INTERACTION OF URBAN STORMWATER RUNOFF, CONTROL MEASURES AND RECEIVING WATER RESPONSE

By

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To My Parents, My Sister, Elena,

and My Fiancée, Margarita

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NOTATION

A	=	wetted cross-sectional area in the physical component, L^2
A_c	=	area served by combined sewers, acres
A_t	=	total area of catchment, acres
AR_u	=	urban area runoff, inches per hour
$A(t)$	=	fluctuating cross-sectional area due to a variable depth, L^2
B	=	benthal demand of bottom deposits, mg/l-hour
BOD	=	Biochemical Oxygen Demand
BOD_c	=	mixed BOD concentration in the combined sewer, mg/l
$\overline{BOD}_{CM}$	=	average field measured BOD concentration in combined sewer overflows, mg/l
BOD_d	=	BOD concentration of dry-weather flow treatment facility effluent, mg/l
BOD_f	=	BOD concentration of municipal sewage, mg/l
BOD_m	=	mixed BOD concentration in receiving water, mg/l
BOD_s	=	BOD concentration of urban stormwater runoff, mg/l

BOD_{sc} = mixed BOD concentration of urban runoff from the separate storm sewer and combined sewer flows, mg/l

BOD_t = hourly BOD concentration of total urban runoff, mg/l

BOD_u = mixed BOD concentration from sources upstream of urban area, mg/l

BOD_w = BOD concentration of treated WWF storage/treatment facility effluent, mg/l

C = concentration of water quality parameters (pollutant), M/L^3

C = concentration of D.O. in the stream, mg/l

c_i = value of input concentration from t_i to $(t_i + \Delta t)$, M/L^3

C_{min} = concentration of D.O. at maximum deficit, mg/l

C_s = dissolved oxygen saturation, mg/l

c.v. = sample coefficient of variation of the water quality parameter

C_o = constant influent concentration, M/L^3

c_1 = factor to convert mg/l to lbs/cf

C_1 = concentration of pollutant in the inflow, dimensionless

c_2 = factor to convert FF from lb/hr to cfs·mg/l

$\overline{C}_2$ = mean effluent concentration, M/L^3

CR_u = composite runoff coefficient dependent on urban land use

$C_o(I)$ = concentration of wastewater in the storage/treatment system at t = 0, for storm event I, M/L^3

$c_2(I-1)$	=	concentration of wastewater in the system during the last hour of runoff of the previous storm event, M/L^3
$C_1(\tau)$	=	concentration forcing function, dimensionless
$C_1(\tau_i)$	=	discrete concentration forcing function, dimensionless
$c_1(t)$	=	concentration forcing function of material in inflow, M/L^3
$c_2(t)$	=	concentration of material in the tank and outflow, as a continuous function of time, M/L^3
$c_2(t_i)$	=	concentration value obtained by equation (5.12) for the previous time step, M/L^3
$\bar{c}_2(t_i + \frac{\Delta t}{2})$	=	time-averaged concentration for each time step, M/L^3
$c_2(x, t - t_1)$	=	pollutant concentration at distance x along the flow axis and time $t \geq t_1$, mass of pollutant/mass of fluid, dimensionless
$C_1(x-Ut, t - \frac{x}{U})$	=	initial concentration at a time x/U earlier and a distance Ut upstream, M/L^3
$D = C_s - C$	=	dissolved oxygen deficit, mg/l
D_c	=	critical (maximum) D.O. deficit, mg/l
D_o	=	initial D.O. deficit, mg/l
D_u	=	D.O. deficit in receiving waters upstream of inflow point, mg/l
D.O.	=	Dissolved Oxygen
DWFSEP	=	DWF contribution from separate sewer area, cfs
DWFCMB	=	DWF contribution from combined sewer area, cfs

DWH	=	number of dry weather hours preceding each runoff event
DWT(I)	=	number of DWH preceding storm event I
E	=	longitudinal dispersion coefficient, L^2/T
$f = K_2/K_1$	=	self-purification ratio, dimensionless
f_u	=	available urban depression storage, inches
FF	=	first flush factor, pounds/DWH-acre
$f(t - \tau)$	=	system step response, dimensionless
g	=	exponent in solution to steady-state differential equation for dissolved oxygen deficit, T^{-1}
H	=	stream depth, feet
h	=	sampling interval = $t_{i+1} - t_i$ = 1 hour
$I(\tau)$	=	system input function, M/L^3
j	=	exponent in solution to steady-state differential equation for dissolved oxygen deficit, T^{-1}
K	=	first-order decay rate of pollutant in the fluid medium, T^{-1}
k	=	number of hourly lags
K_n	=	oxidation coefficient of nitrogenous BOD, $hours^{-1}$
K_1	=	first-order BOD decay constant, day^{-1} and $hours^{-1}$
K_2	=	atmospheric reaeration coefficient, $hours^{-1}$
L	=	length of the storage/treatment basin, feet
L	=	remaining carbonaceous BOD concentration, mg/l
$(L_o)_c$	=	ultimate first-stage BOD demand, mg/l

m = equalized variation of the flow, dimensionless

m = exponent in the steady-state differential equation for dissolved oxygen deficit, L^{-1}

$m = \frac{\Delta t}{\Delta t_2}$ = total number of increments desired per time step

N = remaining nitrogenous BOD concentration, mg/l

N = any desired upper bound, dimensionless number

n = total number of data points or observations

n = total number of inputs

P = oxygen production rate by algal photosynthesis, mg/l-hour

P_u = hourly rainfall/snowmelt in inches over the urban area

Pe = Peclet dispersion number

Q = volumetric flow into and out of tank, L^3/T

$\overline{Q}$ = mean influent fluid flow rate, L^3/T

Q_c = combined sewer overflow rate, cfs

Q_d = DWF treated effluent, cfs

Q_i = value of input flow rate from t_i to $(t_i + \Delta t)$, L^3/T

Q_s = urban runoff carried by the separate storm sewer, cfs

Q_t = total (storm plus combined) urban runoff, cfs

Q_u = upstream flow, cfs

Q_w = WWF storage/treatment effluent, cfs

QCOM	=	total flow generated by the combined sewer area, including DWF contribution, for all periods of urban runoff, cf/yr
$q''(\tau)$	=	variable time rate of mass injection per unit area at the plane source, M/L^2T
$Q_o(I)$	=	system outflow rate at t = 0, for storm event I, L^3/T
q_1	=	influent fluid flow rate per unit width, L^2/T
Q_1	=	influent fluid flow rate, L^3/T
$Q_2(I - 1)$	=	outflow rate from the storage/treatment system during the last hour of runoff of the previous storm event, L^3/T
Q_2	=	outflow rate, L^3/T
$Q_1(t)$	=	fluid flow rate into tank, as a continuous function of time, L^3/T
$Q_2(t)$	=	fluid flow rate out of the tank, as a continuous function of time, L^3/T
$Q_2(t=0)$	=	initial outflow of the system, L^3/T
$Q_2(t_i)$	=	outflow value at the end of the previous time step, L^3/T
$Q_2(t_i + \Delta t)$	=	output flow rate at the end of the time step, L^3/T
$\overline{Q}_2(t_i + \frac{\Delta t}{2})$	=	time-averaged outflow rate for each time step, L^3/T
r	=	total number of hourly runoff occurrences during the year
R_d	=	fraction removal of BOD achieved by the DWF treatment facility
R_e	=	algal respiration rate, mg/l-hour
S	=	sources or sinks of the substance C, M/L^3T

s	=	standard deviation of observations of the water quality parameter from its computed mean
s^2	=	unbiased estimate of the variance of observed magnitudes of the water quality parameter
T	=	water temperature, °C
t	=	Time, T
t_c	=	elapsed time at which the critical deficit occurs, hours
t_D	=	detention time, T
$\overline{t}_D$	=	average detention time, T
t_i	=	the beginning of the time interval for which the system response is being evaluated, T
t_j	=	any time $t \geq t_1$
T_Q	=	correlation time constant for influent fluid flow rates, hours
T_w	=	correlation time constant for mass rate inputs, hours
t_1	=	duration of injection, T
TDWH	=	total number of DWH in the year, DWH/yr
TL (95%)	=	tolerance limits at a 95% probability level
U	=	longitudinal velocity in the storage/treatment system, or in the stream, L/T
V	=	volume of tank, L^3
V(t)	=	time-varying volume of the basin, L^3
W_i	=	value of input mass rate from t_i to $(t_i + \Delta t)$, M/T

$\overline{W}_1$ = mean influent mass rate, M/T

$\bar{w}_2(t_i + \frac{\Delta t}{2})$ = time-averaged output mass rate for each time step, M/T

$\overline{x}$ = unbiased estimate of the mean value of the water quality parameter

x_i = discrete data series (observations) of a hydrologic process I, for i = 1, 2, ..., n

$\overline{X}_w$ = unbiased estimate of the flow-weighted mean value of the pollutant concentration

α = time constant, T^{-1}

α_1 = regression coefficient

α_2 = regression coefficient

β = constant of integration

β_1 = regression coefficient

β_2 = regression coefficient

Δt = length of time interval, say, 1 hour

ν_C = coefficient of variation of the effluent concentration, dimensionless

ν_{CQ} = coefficient of variation of the effluent mass flow rate, dimensionless

ν_Q = coefficient of variation of the influent fluid flow rate, dimensionless

ν_W = coefficient of variation of the influent mass rate, dimensionless

Ω = $\sqrt{U^2+4KE}$, has the dimensions of velocity, L/T

ρ = density of the receiving fluid, M/L^3

σ_Q = standard deviation of the influent fluid flow rate, L^3/T

σ_W = standard deviation of the influent mass rate, M/T

σ_Q^2 = variance of the influent fluid flow rate, L^6/T^2

σ_W^2 = variance of the influent mass rate, M^2/T^2

τ = dummy variable of integration

τ_i = $i\Delta t$, dimensions of time, T

τ_Q = normalized detention time for fluid flow, dimensionless

τ_W = normalized detention time for pollutant mass input, dimensionless

X_i = data series of observed magnitudes of the water quality parameter, for i = 1, 2, 3, . . ., n

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December, 1976

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Major Department: Environmental Engineering Sciences

Pollutant transport within each phase of the hydrologic cycle, and through the various components of the physical system, is governed by the principle of conservation of mass. Deterministic mathematical models are derived from this unifying concept to represent the movement, decay, storage, and treatment of stormwater runoff pollutants and dry weather wastewater flows through the urban environment and the receiving body of water. The general one-dimensional, transient conservation of mass equation may be simplified for application to the various pathways by retention of only the dominant terms in each instance. The detention time and decay coefficient are key parameters in establishing the final form of the governing differential equation.

The precipitation time series and the urban runoff series generated by a continuous hydrologic simulation model are characterized stochastically by autocorrelation analysis to define independent wet weather events. The transient response of storage/treatment systems to variable forcing functions of flow and concentration is determined for completely mixed systems of constant and variable volumes and for one-dimensional advective systems with and without dispersion, and results are compared. Frequency analyses are performed on input and output concentrations and mass rates for a single event and for all wet weather events during the year of record.

The unifying concept of continuity is extended to determine the receiving water response to waste inputs from (1) wet weather urban sources, (2) dry weather urban sources, and (3) upstream sources. The results are presented in terms of minimum dissolved oxygen cumulative frequency curves. Interpretations are based on established stream standards for the study area: Des Moines, Iowa and the Des Moines River. The wet weather flow storage/treatment facility for the receiving water analysis was represented by a well-mixed constant volume model. The receiving water response was found to be sensitive to the length of the detention time in the storage/treatment device, during periods of urban runoff.

CHAPTER I

INTRODUCTION

Historically, scientific concepts have been modes of reasoning rather than absolute realities. In this sense, models are tools of thinking to formulate hypotheses which must be verified by observations (Vollenweider, 1975). The nature of the models themselves depends largely on points of view and objectives. Problems involving physical phenomena seem to fit naturally into three classes (Domenico, 1972):

(1) those studied from a <u>scientific</u> point of view in order to promote a better description of physical phenomena,

(2) those studied from an <u>engineering</u> point of view to achieve useful purposes, such as prediction by using the laws of science, and

(3) problems studied from a <u>management</u> and <u>planning</u> point of view to achieve some degree of control over the state of the system.

Scientific studies are based on the premise that all natural phenomena and processes are interrelated and interactions are governed by certain laws, thus involving numerous subsystems.

The research problem addressed in this work is the interaction of urban stormwater runoff, control measures, and receiving water response. The water quality cycle is a dynamic system interacting with each phase of the hydrologic cycle. The amount of pollution entering or leaving a water body is determined by the quantity of flow and concentration of pollutants in each of the hydrologic components of the physical system. The retention of pollutants in the water body is not a sole function of the quantity and quality of hydraulic flows, since it also depends upon the location of the pollutants within the water body. The pollutants exist in the water, bottom sediments, and the aquatic life.

The engineering problem is finding analytical relationships between the variables characterizing the inflow and outflow processes and parameters defining the state of the system. In principle, these analytic relationships are provided in all cases by solution of the complete equations of energy, mass, momentum, and state (Eagleson, 1970). However, it is seldom possible to formulate all of these equations accurately (Eagleson, 1970) because of

(1) inadequate knowledge of physical behavior,

(2) unknown system heterogeneities and anisotropies,

(3) unknown temporal variabilities of system parameters, and

(4) numerical approximations introduced for computational economy, or to obtain a

solution where direct mathematical analysis is impossible.

Whereas state variables (density, volume, temperature, etc.) define the condition of system components, decision variables act to modify the state. For example, storage and treatment may modify the concentration of pollutants in an accelerated manner to prevent damaging shock loadings from entering receiving bodies of water. The degree of control defines the management problem.

Urban hydrology, including water quality processes, is a combination of concepts and parameters that pertain to scientific, engineering, and management points of view. The essence of a rational approach to water quality control in the urban environment is the development of a conceptual model, based on scientific principles, which has the predictive capabilities necessary to the decision-making process. The high cost of pollution control facilities, in terms of energy utilization, land requirements, engineering manpower, and long-term financial burden, obligates the planning agency to select an optimum strategy for area-wide wastewater management. Such a process must focus on a systematic procedure that identifies and defines (1) the cause/effect relationships of the physical environment, (2) the efficiency of control alternatives, and (3) the benefits to be derived from implementation of these controls. The mathematical models applied need not

incorporate all phenomena but rather should be relevant to the problem under consideration.

Two philosophies have emerged in recent years to provide decision perspectives on urban stormwater pollution: (1) the micromodeling approach, and (2) the macromodeling approach. The more complex urban hydrologic simulation models typify the micromodeling philosophy, where short time increment mathematical models are applied to a specific catchment subjected to a known or synthetic rainfall history. Unquestionably, as the degree of detail increases, so does the difficulty to generalize the results obtained. This is particularly true of sewer routing models, where hydraulic computations depend on diverse sewer cross-sections. For example, 13 different sewer cross-sections are represented in the U. S. EPA Storm Water Management Model (Huber et al., 1975). Thus, varying shapes and dimensions from urban area to urban area will modify the runoff hydrographs. These models are invaluable in identifying problem areas in sewer networks. The Storm Water Management Model can predict the elements where surcharging will occur and, as an option, enlarges the downstream conduit by standard amounts until capacity exists to accept the flow (Heaney et al., 1975). However, experience at the University of Florida in applying the detailed simulation models to large metropolitan areas suggests that the data needs alone may be quite substantial and the

discretization for the mathematical abstraction of the physical drainage system is time consuming (Medina, 1974).

At the other end of the mathematical modeling spectrum is the macroapproach. The methodology is typified by a simplified version of the physical process to compute a scalar quantity which ranks pollution severity and also allows study of the scalar's sensitivity to the interrelationships involved (Young, 1976). The approach focuses on an aggregated receiving water pollutograph, specifically the ultimate BOD concentration versus time. This pollutograph is assumed to represent water quality downstream from an urban area as a flow-weighted average of concentrations from sewage residuals, urban storm washoff and upstream sources. As computed, it is independent of hydrologic rainfall-runoff features, and varies with the receiving water streamflow and the public treatment works of the urban system. Decision rules are based on calculation of scalar pollutograph maxima for ranking the control needs of the various urban areas studied. This approach has considerable merit because of its simplicity of application. However, there are several limitations:

(1) estimates for design streamflow, urban runoff quality, removal efficiencies, and hydraulic capacity of interceptor sewers are largely dependent on the analyst's judgment and experience;

(2) independence of the methodology to variations in volume of runoff and associated pollutants excludes the possibility of evaluating realistically the relative efficiency of control measures and the receiving water response to these measures; and

(3) a single scalar index precludes obtaining any information about the frequency of occurrence of events that result in deleterious water quality conditions in the receiving waterways.

There is a real need for an approach which is intermediate between the micromodeling and macromodeling philosophies, provides for continuous hydrologic simulation, and recognizes that pollutant transport through the various components of the urban environment is governed by the same principle that describes transport in the natural environment--conservation of mass. The justification for continuous hydrologic simulation in dealing with problems of urban storm runoff and urban storm runoff quality is the probability of occurrence of events of various magnitudes (Linsley and Crawford, 1974). Consequently, the research objectives are

(1) to present a unified physical system conceptualization;

(2) to develop a model based on the unifying concept which is representative of the land use, hydrology, and climatology of the drainage area while providing an analytical framework to

-generate stormwater runoff pollutant loads and dry weather sanitary flow pollutant loads,

-simulate the pollutant removal capabilities of various storage/treatment alternatives,

-simulate the conveyance system, including mixing in combined sewers of wet and dry weather pollutants,

-mix the various pollutant inflow combinations with pollutants already in the receiving water (from upstream sources), and

-predict the oxygen balance of the polluted waters downstream from the waste sources,

subject to the constraints imposed by the quantity and quality of the data base; and

(3) to examine, in particular, the response of storage/treatment systems to variable wet weather flows by a detailed mathematical application of the continuity equation.

Scientific principles derived from hydrologic theory, chemical reactor engineering and process dynamics,

unit operations of wastewater engineering, and the mathematics of diffusion are applied to characterize urban runoff quantity and quality, evaluate control measures, and determine receiving water response. A major portion of this work is devoted to storage/treatment systems because of their importance in reducing pollutant loads to the receiving waters from highly variable wet weather flows. These systems, as stated in the objectives, are represented by mass transport models satisfying continuity. Reservoir management models have been proposed using inventory theory (Sobel, 1975), but these models are best suited to formulate release policies which are a function of the quantitative demand for water supply. It is difficult to foresee their application in a context where water quality and interaction between the urban environment and the receiving water are important.

Chapter II discusses the characterization of urban wastewater discharges and polluted surface waters, and presents a unified physical system conceptualization. An oxygen demand parameter is selected to characterize the strength of all waste sources in the system. The principle of conservation of mass is shown to be equally applicable to pipe segments of a sewer system, storage/treatment devices, and receiving bodies of water.

Chapter III describes the study area, including the receiving stream, and presents the necessary modeling

techniques. Typical pollutant loads are summarized and pertinent water quality standards are defined. Data sources are also identified.

Chapter IV presents a stochastic characterization of urban runoff. Techniques of time series analysis are discussed, as well as an approximate graphic procedure, to define a wet weather event.

Chapter V addresses the response of storage/treatment systems to variable inputs of wet weather flows, and their associated pollutant concentrations and mass rates. Storage/treatment units are modeled as well-mixed constant volume systems, well-mixed variable volume systems, and variable volume systems where nonideal mixing is represented by advection and dispersion. Numerical applications are shown for each type of system for various residence times, and statistical summaries of the results are presented.

Chapter VI introduces a statistical approach to storage/treatment response for the case where the system is assumed to be represented by a well-mixed constant volume model. The results are compared to those obtained from solution of the continuity equation in Chapter V. A discussion of the concentration and mass flow rate equalization provided by the well-mixed constant volume model is included.

Chapter VII discusses the methodology adopted for the receiving water analysis. The receiving water response to waste input combinations is presented in the form of minimum dissolved oxygen frequency curves. The interpretation of these results is based on established water quality standards.

Finally, Chapter VIII summarizes the conclusions drawn from this work and the recommendations made for further research.

CHAPTER II

A UNIFIED PHYSICAL SYSTEM CONCEPTUALIZATION

2.1 Characterization of Urban Wastewater Discharges and Polluted Surface Waters

Urban areas represent only 3 percent of total land use in the United States (U. S. Dept. of Agriculture, 1968), yet they constitute the centers of most intense human activity. Point discharges resulting from commercial, industrial, and residential wastes generally enter the receiving stream within relatively short distances of each other, and in some cases all such wastes are processed by municipal facilities and discharged to the water body at one location. Thus, these continuous waste products are concentrated within a relatively small volume of the receiving water. Intermittent precipitation falling on urban areas becomes contaminated as it enters and passes through or within the man-made environment. The first quality degradation occurs when the rainwater comes into contact with pollutants in the air. The Cincinnati Water Research Laboratory of the Robert A. Taft Sanitary Engineering Center started rainfall sampling

in August 1963 (Weibel et al., 1966). Constituents found in the rainfall from August 1963 to December 1964 are listed in Table 2-1. The concentrations represent average values from 35 storms.

Next, the surface runoff passes over ground and building surfaces, carrying suspended sediment from erosion sites and dissolving other impurities. Table 2-2 summarizes typical ranges of pollutant concentrations found in urban stormwater runoff (Field and Tafuri, 1973). Finally, the stormwater runoff comes into contact with (1) solid residues deposited from earlier storms throughout the conveyance system and appurtenances, and (2) dry weather flow (DWF) in combined sewer systems. This storm runoff is well mixed with sanitary sewage under conditions of turbulent flow in a combined sewer system, and it eventually discharges to the receiving stream. Table 2-3 lists ranges of pollutant concentrations found to be typical of combined sewer overflows (Field and Tafuri, 1973). From an examination of Tables 2-1, 2-2, and 2-3, it can be verified that the degradation undergone by urban stormwater passing through the surface runoff phase of the hydrologic cycle (including the collection system) can be several orders of magnitude greater than that experienced by rainwater during the precipitation phase.

Table 2-1

Constituents in Rainfall
(Weibel *et al*., 1966)

Constituent	Concentration
Suspended solids, mg/l	13
COD, mg/l	16
Total N, mg/l (N)	1.27
Inorg. N, mg/l (N)	0.69
Hydrolyzable PO_4, mg/l (PO_4)	0.24
Org. Chlorine*, mg/l	0.28

*From pesticides in the air.

Table 2-2

Characteristics of Urban Stormwater
(Field and Tafuri, 1973)

Constituent	Concentration Range		
BOD_5	1	-	700 mg/l
COD	5	-	3,100 mg/l
TSS	2	-	11,300 mg/l
Total Solids	450	-	14,600 mg/l
Volatile Total Solids	12	-	1,600 mg/l
Settleable Solids	0.5	-	5,400 mg/l
Organic N	0.1	-	16 mg/l
NH_3-N	0.1	-	2.5 mg/l
Soluble PO_4 (as PO_4)	0.1	-	10 mg/l
Total PO_4 (as PO_4)	0.1	-	125 mg/l
Chlorides	2	-	25,000 mg/l*
Oils	0	-	110 mg/l
Phenols	0	-	0.2 mg/l
Lead	0	-	1.9 mg/l
Total Coliform	200	-	146×10^6/100 ml
Fecal Coliform	55	-	112×10^6/100 ml
Fecal Streptococci	200	-	1.2×10^6/100 ml

*With highway deicing.

Table 2-3

Characteristics of Combined Sewer Overflows
(Field and Tafuri, 1973)

Constituent	Concentration Range		
BOD_5	30	-	600 mg/l
TSS	20	-	1,700 mg/l
Total Solids	150	-	2,300 mg/l
Volatile Total Solids	15	-	820 mg/l
pH	4.9	-	8.7
Settleable Solids	2	-	1,550 mg/l
Organic N	1.5	-	33.1 mg/l
NH_3-N	0.1	-	12.5 mg/l
Soluble PO_4 (as PO_4)	0.1	-	6.2 mg/l
Total Coliform	20,000	-	90×10^6/100 ml
Fecal Coliform	20,000	-	17×10^6/100 ml
Fecal Streptococci	20,000	-	2×10^6/100 ml

The various constituents listed so far represent a large and diverse number of chemical, physical, and bacteriological indicators of water quality. The most common parameters measured are listed in Table 2-4 under their appropriate categories. The bacteriological procedures are designed to detect potential health hazards from contamination of the water with human or animal feces. The sampling technique, frequency of sampling, and method of preservation should be tailored to the indicators chosen for measurement. Field observations are extremely valuable for verification of mathematical models. Greater precision may be obtained in the laboratory, but if the sample is unrepresentative, greater accuracy will be achieved by in situ procedures.

The wastewater constituents that affect the distribution of dissolved oxygen (D.O.) in a natural water system are well documented. The oxygen demand of sewage, sewage treatment plant effluent, polluted stormwater runoff or industrial wastes is exerted by three types of materials (American Public Health Association, 1971):

1. carbonaceous organic matter oxidized by heterotrophic bacteria for energy and cell synthesis;
2. organic nitrogen compounds hydrolyzed into ammonia-nitrogen (NH_3-N), then oxidized by autotrophic bacteria (*Nitrosomonas europaea*) to nitrite-nitrogen (NO_2-N), further oxidized

Table 2-4

Pollution and Contamination Indices

Physical Parameters:

- Temperature
- Turbidity
- Color

Chemical Parameters:

- Oxygen Demand
  - Biochemical Oxygen Demand (BOD)
  - Chemical Oxygen Demand (COD)
  - Total Organic Carbon (TOC)
- Nitrogen Compounds
  - Organic
  - Nitrite
  - Nitrate
- Phosphorus Compounds
  - Ortho Phosphorus
  - Poly Phosphates
- Total Solids
  - Dissolved
  - Suspended
  - Volatile and Fixed
  - Settleable
- Chlorides
- Sulfates
- pH
- Alkalinity
- Hardness

Table 2-4 continued

Chemical Parameters continued:
Heavy Metals
Lead
Copper
Zinc
Chromium
Mercury
Biological Parameters:
Plankton
Periphyton
Macrophyton
Macroinvertebrates
Fish Bioassays
Bacteriological Parameters:
Total Coliform Count
Fecal Coliform
Fecal Streptococci
Total Plate Count

by *Nitrobacter winogradskyi* to nitrate-nitrogen (NO_3-N); and

3. certain chemical reducing compounds (ferrous iron, sulfite, and sulfide) which will react with molecularly dissolved oxygen.

This oxygen demand is the response of aquatic biota to an adequate food supply and is commonly referred to as the biochemical oxygen demand (BOD). The laboratory BOD technique is an empirical bioassay-type procedure: the D.O. consumed by microbial life in an incubated bottle is measured with respect to time at a specified temperature. The actual environmental conditions of temperature changes, biological population, water movement, sunlight, and zones of aerobic and anaerobic processes cannot be faithfully reproduced in the laboratory. Thus, the "bottled" system, on a kinetic comparison, is completely accurate in representing itself but may be relatively unreliable as a representation of the source from which the sample was taken. The basic assumption that consumption of D.O. is an absolute and complete parameter of biological decomposition in the BOD bottle constitutes a simplification of complex interactions (Liptak, 1974).

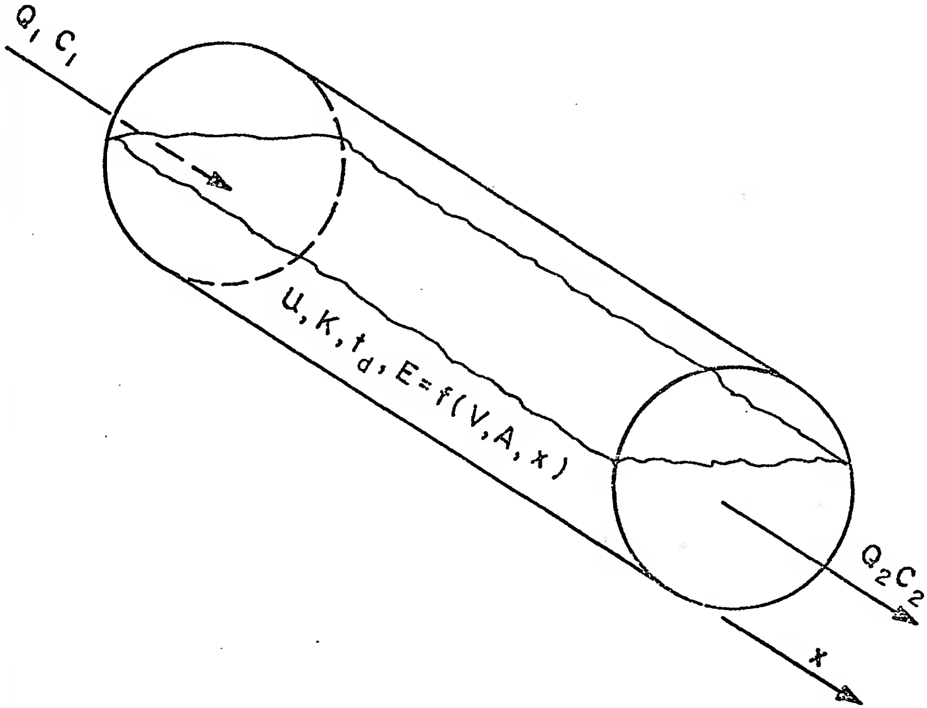
Other laboratory methods have been developed to measure the oxygen demand exerted by organic matter. The chemical oxygen demand (COD) and total organic carbon (TOC) tests are more precise _chemical_ methods, but the

analytical results are not accurate if the organic material measured is not equivalent to the organic matter actually being utilized by the microorganisms in the stream. Much can be done to improve the accuracy of the BOD test by using dilution water from the receiving stream, thus introducing a natural "seed" of diverse organisms into the bottle system. With all of its limitations, the BOD procedure is still considered to be the best method for evaluating the effect of waste inputs on the oxygen balance of a stream (Nemerow, 1974).

2.2 A Unifying Conceptualization of the Physical System

The principle of conservation of mass may be applied universally throughout the urban and natural environments to describe the transport of pollutants. Figure 2-1 represents a generalized component of the physical system, which may characterize (1) a pipe segment of the sewer system, (2) a storage/treatment device, or (3) a reach of the receiving body of water. Essentially, all of these subsystems may be approximated by the one-dimensional version of the convective-dispersion equation,

$$\frac{\partial C}{\partial t} = \frac{\partial}{\partial x}\left[E\frac{\partial C}{\partial x} - UC\right] \pm \Sigma S \qquad (2.1)$$



$$\frac{\partial c}{\partial t} = \frac{\partial}{\partial x}\left[E\frac{\partial c}{\partial x} - uc\right] \pm \sum S$$

Figure 2-1. A Generalized Component of the Physical System.

where C = concentration of water quality parameter (pollutant), M/L^3,

t = time, T,

$-E\frac{\partial C}{\partial x}$ = mass flux due to longitudinal dispersion along the flow axis, the x direction, M/L^2T,

UC = mass flux due to advection by the fluid containing the mass of pollutant, M/L^2T,

S = sources or sinks of the substance C, M/L^3T,

U = flow velocity, L/T, and

E = longitudinal dispersion coefficient, L^2/T.

The source/sink term accounts for biochemical processes (e.g., decay, photosynthesis, algal respiration), boundary losses such as stream benthic deposits, and boundary gains (e.g., reaeration, and point or distributed waste discharges). Assuming that the longitudinal dispersion coefficient and the advective velocity are constant along the flow axis, equation (2.1) may be expanded for the generalized component in Figure 2-1 to

$$\frac{\partial C}{\partial t} = E\,\frac{\partial^2 C}{\partial x^2} - U\,\frac{\partial C}{\partial x} - KC + \frac{q_1}{A}(C_1 - C) \qquad (2.2)$$

where K = first-order reaction rate coefficient, $1/T$,

q_1 = influent fluid flow rate per unit width, L^2/T,

C_1 = concentration of water quality parameter in the inflow, M/L^3, and

A = wetted cross-sectional area in the component, L^2.

In equation (2.2) the influent fluid flow rate per unit width, influent concentration, and the wetted cross-sectional area may all be variable functions of time.

For a nondispersive pipe segment, equation (2.2) may be reduced to the form for simple advection.

$$\frac{\partial C}{\partial t} + U\frac{\partial C}{\partial x} + KC = 0 \tag{2.3}$$

The solution to equation (2.3) is given by

$$C(x,t) = C_1(x-Ut,\ t - \frac{x}{U})\exp[-Kt] \tag{2.4}$$

where $C_1(x-Ut,\ t - \frac{x}{U})$ = initial concentration at a time x/U earlier and a distance Ut upstream, M/L^3.

Equation (2.4) states that the reactive, nondispersive advective system has an instantaneous response which travels downstream at a velocity U, and the magnitude of the response decreases exponentially because of first-order decay. Chemical engineers refer to a system governed by equation (2.3) as a plug flow reactor. If

there is no decay, or if it can be assumed that it is negligible because the travel time in the system is very short (a large U), then equation (2.4) reduces to

$$C(x,t) = C_1(x - Ut,\ t - \frac{x}{U}) \qquad (2.5)$$

This is a reasonable assumption to make for most sewer systems when the study objectives are not design-oriented.

Extensive studies have been conducted by chemical engineers concerning the kinetic analysis, flow characteristics, and dispersion characteristics of chemical process reactors. Danckwerts (1953) studied the residence-time distribution of continuous-flow systems. The basic differential equation for a continuous-flow reactor is given as

$$E \frac{\partial^2 C}{\partial x^2} - U \frac{\partial C}{\partial x} - KC = 0 \qquad (2.6)$$

for a first-order reaction. Equation (2.6) is obtained from equation (2.2) for steady-state conditions ($\partial C/\partial t = 0$). Wehner and Wilhelm (1956) solved equation (2.6) analytically. Their solution has been recommended by Thirumurthi (1969) for the design of large waste stabilization ponds, aerated lagoons, and activated sludge aeration tanks.

For completely mixed storage/treatment systems ($\partial C/\partial x = 0$), equation (2.1) reduces to

$$\frac{\partial C}{\partial t} = \Sigma\ S \tag{2.7}$$

which may be rewritten, for the sources and sinks of the generalized component in Figure 2-1, as the ordinary differential equation

$$V \frac{dC}{dt} = Q_1 C_1 - Q_2 C - KC\ V \tag{2.8}$$

where Q_1 = influent fluid flow rate, L^3/T,

Q_2 = outflow rate, L^3/T, and

V = volume of the fluid mass in the component, L^3.

Solutions to equation (2.8) are presented in Chapter V for the constant volume and variable volume cases, for time-varying influent flow rates and concentrations. In chemical engineering texts (e.g., Cooper and Jeffreys, 1971), the design equation for a continuous-flow stirred tank reactor is given as

$$\frac{C}{C_o} = \frac{1}{1 + Kt_D} \tag{2.9}$$

where C_o = constant influent concentration, M/L^3, and

t_D = detention time in the reactor, T.

This equation is used by Eckenfelder (1966) in designing aerated lagoons for industrial water pollution control. It should be noted that equation (2.9) predicts the steady-state value of the general solution to equation (2.8) for a constant volume system. A methodology is also presented in Chapter V to obtain solutions to equation (2.1) for the more difficult case of a dispersive variable volume system.

A receiving body of water, such as a river reach or an estuary, is governed by equation (2.1). Whereas C represented the BOD concentration of wastewaters in the urban environment, equation (2.1) is usually solved to obtain the dissolved oxygen concentration in the natural system. The BOD inputs from the urban area become oxygen sinks, along with benthal demand, respiration by aquatic plants, and others. An oxygen concentration gradient is developed between the atmosphere and the waterway, leading to reaeration as a dissolved oxygen source. This process is supplemented by algal photosynthesis. A steady-state analysis of equation (2.1) is presented in Chapter VII.

As noted in Figure 2-1, all of the systems described by the generalized component are essentially characterized by

(1) a flow velocity U,

(2) a reaction rate coefficient K,

(3) a residence time t_D, and

(4) a longitudinal dispersion coefficient E,

all of which are a function of the system geometry (volume V, cross-sectional area A, length x). Velocities in the sewer systems tend to be much higher than in either the storage/treatment devices or the receiving waters. In a receiving stream reach, lake or bay the detention time is much longer than in the other systems due to the large volume of the fluid mass relative to the wastewater inflows. From performance-time curves of activated sludge units, an overall BOD first-order decay coefficient of 0.2 per hour (4.8/day) may be obtained for the biological waste treatment process (Fair, Geyer and Okun, 1968). For tidal rivers and estuaries a first-order decay rate for BOD not much higher than 0.02 per hour (0.5/day) can be expected (Hydroscience, 1971). In lakes, K may vary from 0.01/day for the dry season to 0.02/day for the wet season (Huber et al., 1976), and the detention time is on the order of months. By contrast, a detention time of 10 to 11 hours is common design practice for conventional activated sludge plants (American Society of Civil Engineers, 1959). Estimates of K, t_D, and the product Kt_D are summarized for various systems in Table 2-5. The reaction rate group Kt_D provides a rough idea of treatment potential.

Table 2-5

Treatment Potential of Various Components of the Physical System

Component	Selected Decay Coefficient K at 20°	Selected Detention Time, t_D	Kt_D
Pipe Segment[a] of Sewer System	0.005/hour	0.56 hours	0.003
Activated-Sluge Plant for Dry Weather Flow	0.2/hr	11 hours	2.20
Waste Stabilization Pond[b] for Raw Sewage	0.061/day	39.8 days	2.40
Wet Weather Flow Storage/ Treatment System	0.005/hour	24 hours	0.12
Kissimmee River[c]	0.3/day	9 days	2.70
Lake Toho[d]	0.015/day	5 months (150 days)	2.25
Tidal River or Estuary	0.5/day	5 to 50 days	2.5 to 25.0

[a]Segment length 10,000 feet (3 km) with mean velocity of 5 ft/sec (1.5 m/sec).

[b]Average of five ponds in Fayette, Missouri (Thirumurthi, 1969).

[c]Kissimmee River, Florida for a length of 90 (144 km) from Huber et al., 1976; and Hydroscience, 1971.

[d]Lake Toho, Florida, decay rate for total phosphorus, from Huber et al., 1976.

Comparisons based on the reaction rate group Kt_D in a dispersive system must be qualified. For example, the longitudinal dispersion coefficient in the aeration tank of an activated sludge process may be expected to be around 1.6 ft^2/sec (Murphy and Timpany, 1967). For a tidal river or estuary, about 300 ft^2/sec represents the lower limit of the longitudinal dispersion coefficient. The differences in mass flux due to advection (U) and dispersion (E) will vary the system response considerably. Additional discussion of the importance of these parameters is presented in Chapter V. Furthermore, bays and lakes may be treated in a detailed manner by a multi-dimensional form of equation (2.1), namely, a 2-dimensional or 3-dimensional expression of the convective diffusion equation. Crude approximations may be obtained by simply applying equation (2.8), which assumes completely mixed systems.

CHAPTER III

DESCRIPTION OF THE STUDY AREA AND NECESSARY MODELING TECHNIQUES

3.1 The Need for a Study Area

Pitt and Field (1974) applied available information on urban runoff pollution to a hypothetical test area, subjected to a worst-case storm event, to demonstrate potential problems and their solutions. However, it has already been established that there is a need for continuous hydrologic simulation to assess the frequency with which runoff events cause adverse effects in the receiving waters. It would be quite difficult to generate a realistic pollutant distribution for a long sequence of synthetic flows. There are other important reasons that justify selection of a real study area. The only way to establish the necessary validity which renders a mathematical model of water quality useful for planning purposes is to conduct verification procedures and calibrate against field-measured data (Hydroscience, 1971). Furthermore, when evaluating the effectiveness of proposed control measures it is important to base comparisons against existing conditions in the study area.

3.2 Des Moines, Iowa

The city of Des Moines, Iowa is located at the confluence of the Des Moines River and the Raccoon River as shown in Figure 3-1. Within its limits are approximately 200,000 people out of a total of 288,000 for the metropolitan area. The mean annual precipitation is about 31 inches (787 mm), approximately equal to the State of Iowa average and the United States average (Waite, 1974). The urban area covers 49,000 acres (19,830 ha) of land which has gently rolling terrain. Most of the area, 45,000 acres, is served by separate sewers, while 4,000 acres are served by combined sewers.

Selection of the study area was based primarily on data availability. Davis and Borchardt, of Henningson, Durham & Richardson, Inc., Omaha, Nebraska, conducted an extensive sampling program of combined sewer overflows, stormwater discharges, and surface waters in the Des Moines, Iowa Metropolitan Area for the U. S. Environmental Protection Agency. The objective was a combined sewer overflow abatement plan for the metropolitan area. The sampling program was conducted from March 1968 to October 1969. Other considerations revolved around the fact that Des Moines, Iowa is somewhat typical of many urban centers throughout the country.

(1) it has a medium-sized population;

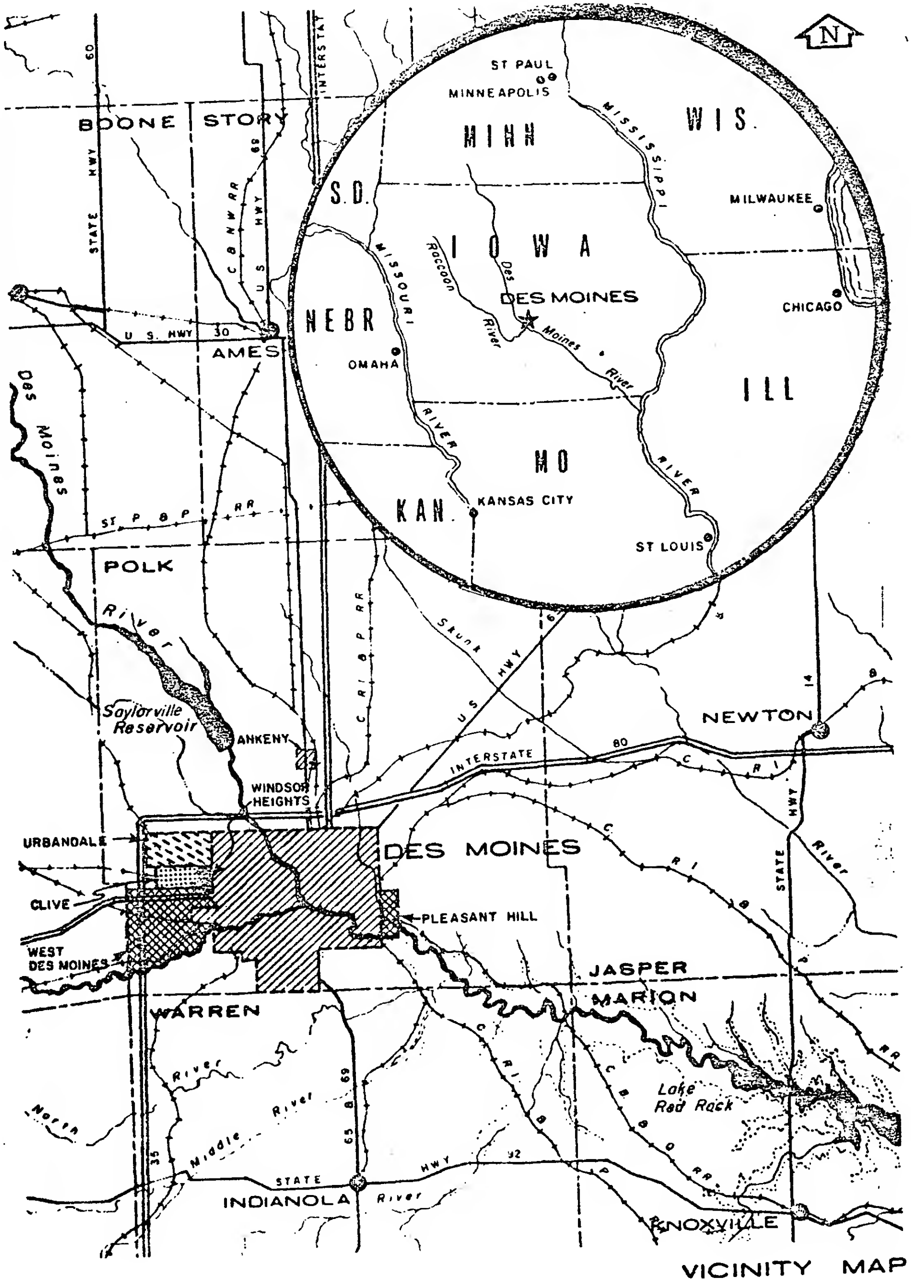


Figure 3-1. Map of Des Moines Area (Davis and Borchardt, 1974).

(2) its domestic and industrial dry weather flows receive secondary treatment;

(3) its wastewaters are discharged into a non-tidal receiving stream; and

(4) the urban area receives a mean annual precipitation equal to the national average.

The mean annual temperature is 50°C (10°C), varying from an average of 21°F (-6°C) in January to an average of 75°F (24°C) in July.

3.3 Pollutant Loads

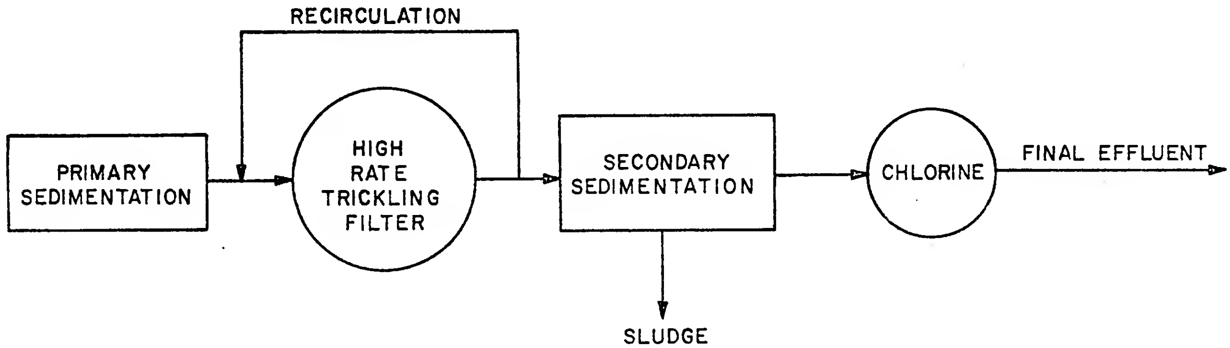
Annual pollutant unit loads upstream from the city are summarized in Table 3-1, as determined by Davis and Borchardt (1974). From data gathered by wastewater treatment plant personnel during the period August to October, 1969, the average daily flow was estimated at 35.3 mgd (133,600 cu m/day) with an organic load from domestic and industrial sources of 85,800 pounds of BOD (43,450 kg BOD per day). The high-rate trickling filter facility achieved 84 to 85 percent BOD removal, producing an effluent BOD concentration of 49 mg/l (Office of Water Planning and Standards, 1974). A simplified profile of the DWF treatment facility is shown in Figure 3-2. The estimated annual loading from the urban area's separate and combined sewer systems is presented in Table 3-2.

Table 3-1

Pollutant Unit Loads for Drainage Area*
Above Des Moines, Iowa
(Davis and Borchardt, 1974)

	Des Moines River	Raccoon River	Total
Drainage Area, acres (ha)	3,738,000 (1,512,769)	2,202,000 (891,149)	5,940,000 (2,403,918)
Unit Average Annual Runoff, acre-ft/acre (ha-m/ha)	0.42 (0.13)	0.40 (0.12)	0.41 (0.12)
Unit BOD, lbs/acre (kg/ha)	13.40 (15.02)	6.93 (7.77)	11.01 (12.34)
Unit NO_3, lbs/acre (kg/ha)	3.75 (4.20)	3.74 (4.19)	3.75 (4.20)
Unit PO_4, lbs/acre (kg/ha)	0.54 (0.61)	0.42 (0.47)	0.50 (0.56)

*On an annual basis.



DESIGN FLOW: 35.3 MGD

POP. SERVED: 247,920

BOD INFLUENT: 325 mg/l

BOD EFFLUENT: 49 mg/l

% BOD REMOVAL: 85

Figure 3-2. Existing DWF Process Profile.

Table 3-2

Summary of Present Annual Metro Area Discharges[a]
(Davis and Borchardt, 1974)

	Days	BOD, lbs (kg)	NO_3, lbs (kg)	$O.PO_4$, lbs (kg)
WWTP Effluent				
Dry Weather	257	4,060,600 (1,841,857)	400,900 (181,845)	1,737,300 (788,026)
"Wet" Dry Weather[b]	108[e]	2,246,400 (1,018,950)	237,600 (107,774)	1,036,800 (470,285)
Subtotal	365	6,307,000 (2,860,807)	638,500 (289,619)	2,774,100 (1,258,311)
"Wet" Dry Weather Overflow[c]	108[3]	2,235,600 (1,014,051)	9,700 (4,400)	263,500 (119,522)
Wet Weather Combined Sewer Overflows[d]				
2.72 in.(69.1 mm) Rain	1	40,500 (18,370	240 (109)	6,350 (2,880)
1.50 in.(38.1 mm) Rain	5	101,500 (46,040)	680 (308)	12,200 (5,534)
0.75 in.(19.1 mm) Rain	12	32,500 (14,742)	220 (100)	3,250 (1,474)
0.375 in. (9.5 mm) Rain	18	0	0	0
0.175 in. (4.4 mm) Rain	20	0	0	0

Table 3-2 continued

	Days	BOD, lbs (kg)	NO_3, lbs (kg)	$O.PO_4$, lbs (kg)
Wet Weather Combined Sewer Overflows[d] (continued)				
Subtotal	56	174,600 (79,197)	1,140 (517)	21,800 (9,888)
Urban Storm Water Discharges[d]				
2.72 in. (69.1 mm) Rain	1	292,000 (132,449)	6,800 (3,084)	3,900 (1,769)
1.50 in. (38.1 mm) Rain	5	765,000 (346,998)	15,300 (6,940)	9,200 (4,173)
0.75 in. (19.1 mm) Rain	12	966,000 (438,170)	19,300 (8,754)	12,000 (5,443)
0.375 in. (9.5 mm) Rain	18	495,200 (224,619)	9,900 (4,491)	6,200 (2,812)
0.175 in. (4.4 mm) Rain	20	149,800 (67,948)	3,000 (1,361)	1,900 (862)
Subtotal	56	2,688,000 (1,219,256)	54,300 (24,630)	33,200 (15,059)
Total Annual Discharge	365	11,385,100 (5,164,194)	703,640 (319,166)	3,092,600 (1,402,780)

[a]Based on sampling periods from October, 1968, to October, 1969.

[b]Based on hydraulic capacity of treatment plant and its average effluent BOD during wet weather periods from May to July, 1969.

[c]Refers to sanitary overflow during wet weather.

[d]Based on annual average rainfall patterns obtained from NOAA.

[e]Includes severe infiltration which prolonged effects of wet weather.

Taking the total upstream drainage area for the Raccoon and Des Moines Rivers, the pollutant contributions are 65,225,000 pounds of BOD (29,586,000 kg); 22,222,000 pounds of NO_3 (10,080,000 kg); and 2,940,000 pounds of PO_4 (1,334,000 kg). The urban area loadings (when added to upstream values) represent, respectively, 15 percent, 3 percent and 51 percent of the total BOD, NO_3, and PO_4 mass loadings to the Des Moines River below the metropolitan area. The Davis and Borchardt estimates, made from river sampling data taken below Des Moines, indicate the following average annual river loadings: 70,000,000 pounds of BOD (31,851,466 kg); 25,400,000 pounds of NO_3 (11,521,250 kg), and 7,950,000 pounds of PO_4 (3,606,059 kg). These figures reveal that (1) 6,610,000 pounds of BOD (2,998,246 kg) are "lost" in transit through the urban section of the stream, and (2) by contrast 2,474,360 pounds of NO_3 (1,122,351 kg) and 1,917,400 pounds of PO_4 (869,718 kg) are gained in addition to the measured urban sources.

The authors of the report offer some explanations.

> The 'sometimes' decreases in organic load through the metro area may be attributable to treatment realized in the low head impoundments at Scott and Center Streets on the Des Moines River and just below Fleur Drive on the Raccoon. To some extent these impoundments may be serving as intermittent sedimentation and stabilization units.
>
> All BOD data, including those used from the two other studies, were obtained

> from unfiltered samples. However, since the analytical technique was the same for all samples, the relative magnitude of the data should not be affected.
>
> There has been some speculation that treated wastewater effluents may exert an antagonistic or retardant effect on the BOD exertion rate of the receiving stream. If true, this may be due to surfactants or to the expected lower exertion rate of the effluent. In this regard, the decreased BOD in 4 or 5 measurements between R-5 and R-6 is of interest. Increased loads between the summation of R-4 and R-9 versus R-5 are likely due to raw and combined sewage bypassing the intervening area.
>
> Another, and probably the most practical, possibility for the discrepancies is the fact that the data are biological and biochemical in nature and such data do not always provide predictable comparative summations. (p. 108)

The river sampling stations (R-4, R-5, R-6 and R-9) are shown in Figure 3-3. It is possible that intervening creeks, such as Beaver Creek which carries nutrient loads of 2,860,000 pounds of NO_3 per year (1,297,274 kg per year) and 390,000 pounds of PO_4 per year (176,900 kg per year), may answer observed differences in the nitrate loads. However, the phosphate totals remain unbalanced and the cause unresolved.

Combined sewer overflows were monitored at five locations and stormwater discharges at three locations. The average BOD concentration of stormwater was reported as 53 mg/l, while the average BOD concentration in combined sewer overflows was 72 mg/l.

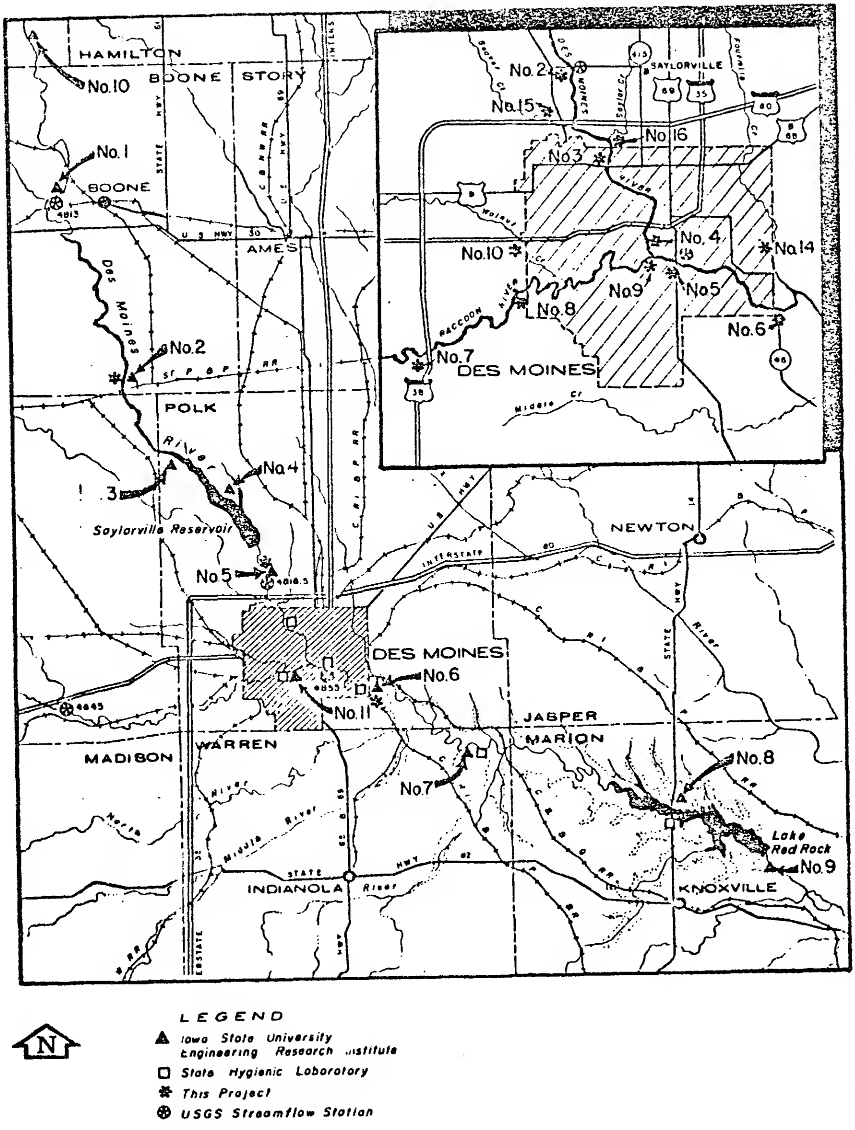


Figure 3-3. Location Map: River Sampling Points (Davis and Borchardt, 1974).

3.4 The Receiving Stream

From Figure 3-3, it is clear that although the Raccoon River receives urban stormwater discharges from parts of the urban area, the ultimate recipient is the Des Moines River. All stormwater discharges, combined sewer overflows, and wastewater treatment plant effluent are assumed to enter the Des Moines River at its confluence with the Raccoon River. The Des Moines River stretches for 200 miles (322 km) from the City of Des Moines to its junction with the Mississippi River and, in general, the river is wide and swift with occasional deep holes and a broad flood plain. According to the State Hygienic Laboratory (1974) bottom material is composed of silt deposits, sand, gravel and rubble providing numerous habitats for fish and other aquatic life. Recreational activities such as fishing and boating are quite heavy. The entire reach is classified as warm water "B" stream by the Iowa Water Quality Standards, such that the absolute minimum dissolved oxygen level specified is 4.0 mg/l. The Iowa Standards also require a minimum of 5.0 mg/l during at least 16 hours per day (State Hygienic Laboratory, 1970).

For the heavy precipitation months of June, July and August, 1968, Davis and Borchardt reported the following nutrient concentrations at a point approximately 5.5

miles (9.0 km) downstream from the confluence of the Raccoon and Des Moines Rivers:

(1) total organic nitrogen ranged from 1.6 to 3.7 mg/l,

(2) nitrate nitrogen ranged from 0.2 to 7.8 mg/l, averaging 3 to 4 mg/l, and

(3) orthophosphate ($O\text{-}PO_4$) ranged from 0.6 to 1.8 mg/l, averaging slightly over 1.0 mg/l.

Davis and Borchardt also observed high algal densities in both the Des Moines and Raccoon Rivers, and state that nutrient concentrations are almost always present at levels reported by Sawyer (1966) to be sufficient for nuisance algal growths: 0.3 mg/l for inorganic nitrogen (NH_3, NO_2, NO_3) and 0.015 mg/l of inorganic phosphorus.

3.5 An Abstraction of the Physical System

To study the interaction of urban runoff, control measures, and receiving water response, an analytic framework is sought that describes the key elements of the physical system. Conceptual models are normally expressed in mathematical terms (Vollenweider, 1975). A conceptual model is derived from an abstraction of the physical system, and such an image is presented in Figure 3-4.

The urban community served by a separate sewer system will convey stormwater runoff and municipal sewage through conduits which are not connected together. The

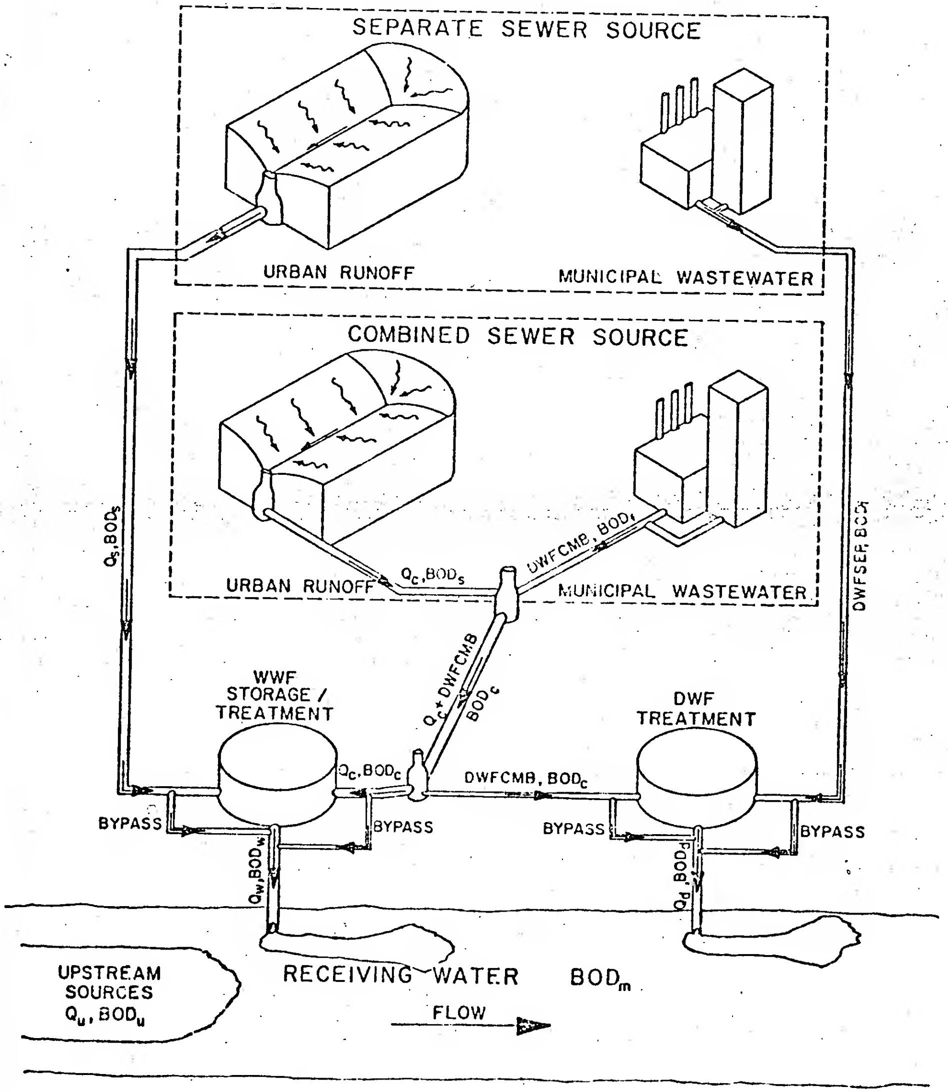


Figure 3-4. An Abstraction of the Physical System.

BOD concentration of the storm sewer runoff is mixed with the dry weather flow (DWF) and accumulated sewer solids. An interceptor carries the sanitary design flow to the municipal sewage treatment plant. Since complete mixing is assumed, the BOD concentrations of the combined sewer overflow and the flow intercepted for treatment by the DWF plant are identical. The combined sewer overflow and the separate storm sewer flow are both processed through a central wet weather flow (WWF) storage/treatment facility before discharge to the receiving body of water.

An examination of Figure 3-4 from top to bottom reveals three distinct phases of the water quality cycle within the urban environment. At the top level is the <u>pollutant build-up</u> phase, represented by surface runoff generated within the separate and combined sewer areas as well as industrial and domestic wastewater. Estimation of DWF sanitary loads is a relatively simple task. The design flow for a sanitary wastewater treatment plant is based on water consumption within the urban area, and the organic waste load is a function of population density and data obtained from industrial and commercial establishments. However, the estimation of surface runoff quantity and quality is more complex.

Urban runoff is a direct result of precipitation over an urban area. However, due to temporal and spatial variabilities associated with atmospheric motions, all

hydrologic processes are more or less stochastic (Chow, 1964). If a deterministic system receives a random input, output from the system will also be random. A single time history representing a random phenomenon is referred to as a <u>sample record</u> when observed over a finite time interval (Bendat and Piersol, 1971). By selecting a sample record of point precipitation, a random element in the hydrologic process is eliminated: chance of occurrence. The precipitation time series may then be fed into an urban hydrologic simulation model to generate the urban runoff series. The spacing and sizing of individual events in the sequence, however, is probabilistic (Eagleson, 1970). Therefore, a combination of probabilistic and deterministic methods must be used. At this stage, analysis of <u>time series</u>, a classical statistical technique, is applied to define a minimum interevent time and group runoff occurrences into independent storm events. A detailed presentation is made in Chapter IV.

The second phase in the urban water quality cycle is at the middle level in Figure 3-4, represented by the man-made <u>conveyance</u> and <u>pollutant control</u> systems. Since travel times in the sewer system are usually relatively short, it has been assumed that no pollutant decay occurs in the conduits. The BOD concentration of the DWF treatment plant effluent is given by

$$BOD_d = \frac{[BOD_f \cdot DWFSEP + BOD_c \cdot DWFCMB]\ (1-R_d)}{DWFSEP + DWFCMB} \tag{3.1}$$

where BOD_f = BOD concentration of municipal sewage, mg/l,

BOD_c = mixed BOD concentration in the combined sewer, mg/l,

DWFSEP = DWF contribution from separate sewer area, cfs,

DWFCMB = DWF contribution from combined sewer area, cfs, and

R_d = fraction removal of BOD achieved by the DWF treatment facility.

Treatment of the DWF facility is accomplished in a single time step. That is, a mass of pollutant entering the DWF plant at a particular time step is treated and released during that time step, with no effects extending beyond the discrete time interval. The effects of storage/treatment in the WWF facility are dynamic and extend beyond the time step length, depending on the detention time primarily. The BOD concentration of the input to the WWF unit is given by

$$BOD_{sc} = \frac{BOD_s \cdot Q_s + BOD_c \cdot Q_c}{Q_s + Q_c} \tag{3.2}$$

where BOD_{sc} = mixed BOD concentration of urban runoff from the separate storm sewer and combined sewer flows, mg/l,

BOD_s = BOD concentration of urban stormwater runoff, mg/l,

Q_s = urban runoff carried by the separate storm sewer, cfs, and

Q_c = combined sewer flow below the interceptor, and to the WWF storage/treatment units, cfs.

To characterize the response of the WWF storage/treatment system, mass transport models derived from the concepts of continuity presented in Chapter II are applied to the wet weather quantity and quality inputs. The complete mathematical analysis is presented in Chapter V. Frequency analyses are performed on concentration and mass rate inputs, and outputs from selected models of the storage/treatment system.

At the bottom level of Figure 3-4 is the third phase of the water quality cycle in the urban environment: the natural pollutant control system provided by the waste assimilation capacity of the receiving body of water. An analogy has been drawn in Chapter II between the man-made storage/treatment system discussed above and the receiving water. Both systems are governed by the same principles of conservation of mass. The mass of pollutant entering either body is subjected to

(1) mixing with the impurities already present in the system within a specified volume;

(2) first-order decay of nonconservative substances;

(3) mass flux due to longitudinal dispersion along the flow axis; and

(4) mass flux due to advection along the flow axis by the fluid containing the pollutant.

The concentration of the combined BOD inputs in the receiving water is given by

$$BOD_m = \frac{BOD_u Q_u + BOD_d Q_d + BOD_w Q_w}{Q_u + Q_d + Q_w} \tag{3.3}$$

where BOD_m = mixed BOD concentration in receiving water, mg/l,

BOD_u = mixed BOD concentration from sources upstream of urban area, mg/l,

BOD_d = BOD concentration of dry weather flow treatment plant effluent, mg/l,

BOD_w = BOD concentration of WWF storage/treatment facility effluent, mg/l,

Q_u = upstream flow, cfs,

Q_d = DWF treated effluent, cfs, and

Q_w = WWF storage/treatment effluent, cfs.

The initial conditions of BOD in the river are defined by equation (3.3), and the hypothetical impact on the oxygen balance of the receiving stream is estimated using simplified mathematical modeling approaches. Critical D.O. deficits are computed as a function of several stream parameters: temperature, flow, oxygen concentration,

deoxygenation and reaeration rates, BOD concentrations, velocity, and the longitudinal dispersion coefficient. Minimum DO's are calculated subsequently and frequency analyses are performed.

3.6 Technique for Calculation of Urban Runoff Quantity and Quality

This section briefly describes the methods used to generate storm runoff and pollutant concentrations. The Hydrologic Engineering Center model, STORM, is utilized to obtain hydrographs and pollutographs for Des Moines for the year 1968 on an hourly time step.

Urban Runoff Quantity

STORM computes urban runoff as a function of land use and rainfall/snowmelt losses (Hydrologic Engineering Center, 1975).

$$AR_u = CR_u(P_u - f_u) \qquad (3.4)$$

where AR_u = urban area runoff, in/hr,

CR_u = composite runoff coefficient dependent on urban land use,

P_u = hourly rainfall/snowmelt in inches over the urban area, and

f_u = available urban depression storage, in.

A maximum depression storage of a hundredth of an inch (0.25 mm) is assumed for Des Moines, Iowa. The hourly urban runoff values, expressed in cfs, are saved in a file for later recall by the simplified mathematical model.

Urban Runoff Quality

The basic water quality parameters modeled by STORM are suspended and settleable solids, BOD, total nitrogen (N), and total phosphate (PO_4). It is important to emphasize that the BOD values are expressed in terms of the standard BOD_5 test: incubation at a temperature of 20°C for five days. These values represent most of the carbonaceous oxygen demand exerted by organic matter present in the urban runoff, and include the BOD contribution from suspended and settleable solids. The BOD loading rates generated by STORM are based on land use and other factors such as number of dry days without runoff since the last storm and the street sweeping intervals.

3.7 Separate, Combined and Dry Weather Loading

All of the following methodology can be used, regardless of the technique employed to generate storm

runoff and quality, as long as these values pertain to the entire area being modeled.

Separate Flows and Loadings

Apportionment of the total flow and BOD loading is made on the basis of the relative area served by separate and combined sewers. Runoff from separate sewered areas is thus (refer to Figure 3-4)

$$Q_s = \frac{A_s}{A_t} Q_t \tag{3.5}$$

where Q_s = stormwater flows from separate sewered areas, cfs,

A_s = area served by separate sewers, acres,

Q_t = total (storm plus combined) urban runoff, cfs, and

A_t = total area of catchment, acres.

The concentration of BOD in separate storm sewers, BOD_s, is simply the hourly value computed by STORM, BOD_t (mg/l), for the total urban runoff.

Dry Weather Flow and Loadings

Dry weather flow and BOD loadings are assumed known from data on point sources in the area. Thus, Q_d represents

the flow (cfs) into receiving waters of treated wastewater, and BOD_d represents the BOD concentration at 68°F (20°C) for five days, mg/l. The amount of treatment can be varied in the analysis.

<u>Combined Flows and Loadings</u>

Dry weather flow is assumed to cause only a negligible increase in flow in a combined sewer during a storm event. However, two factors related to DWF may increase significantly the BOD concentration of the combined sewer storm water.

(1) The BOD strength of the municipal sewage with which it mixes, and

(2) the BOD exerted by sediment accumulation in each section of the sewer under DWF conditions which is subject to the "first flush" effect induced by the initial runoff.

Data collected at various combined sewer overflow sampling stations in Des Moines, Iowa, support the first flush theory (Davis and Borchardt, 1974). BOD and total suspended solids (TSS) concentrations decreased with time with little or no relation to the flow pattern. Furthermore, pollutographs (BOD vs. time and TSS vs. time) for these stations seem to indicate that the flushing occurs

mostly during the <u>first hour</u> of runoff generated by the storm event. A model based on shear stress considerations was developed to predict dry weather flow deposition for 30 combined sewer collection systems in Dorchester, Massachusetts (Pisano, 1976). The overall percentage of DWF suspended solids loadings depositing daily in the systems was estimated at 10.3 percent.

To incorporate the first flush effect, it is assumed that the hourly in-sewer sediment build-up is constant over consecutive dry weather hours (DWH). This assumption is reasonable although particle size and specific gravity, depth of flow and the slope of the conduit are important factors affecting deposition. The computation of combined sewer flows, as with separate sewer flows, is accomplished by apportionment of the total hourly flow predicted by the urban hydrologic model on the basis of relative area served.

$$Q_c = \frac{A_c}{A_t} Q_t \tag{3.6}$$

where A_c = area served by combined sewers, acres, and

Q_c = combined sewer overflow rate, cfs.

The BOD load contributed from accumulation of sewer solids during consecutive DWH is lumped into

the first hour of a runoff sequence, and is estimated by

$$FF = FFLBS \cdot DWH \tag{3.7}$$

where FF = first flush BOD load, lbs,

$FFLBS$ = first flush factor, lbs/DWH, and

DWH = number of dry weather hours preceding each runoff event.

The first flush factor is site-dependent and must be determined from (1) the total flow generated by the combined sewer area, including the DWF contribution, for all periods of urban runoff during the year; (2) the total number of DWH during the year; and (3) the average field measured BOD concentration in the combined sewer overflows. For Des Moines, Iowa, it was estimated from digital computer simulation that about 14 percent of the average BOD load in combined sewer overflows may be attributed to first flush effects. Thus, the first flush factor, in pounds of BOD per DWH, may be obtained from

$$FFLBS = \frac{0.14\ \overline{BOD}_{CM} \cdot QCOM \cdot c_1}{TDWH} \tag{3.8}$$

where $\overline{BOD}_{CM}$ = average field measured BOD concentration in combined sewer overflows, mg/l,

QCOM = total flow generated by the combined sewer area, including DWF contribution, for all periods of urban runoff, cf/yr,

TDWH = total number of DWH in the year, DWH/yr, and

c_1 = factor to convert mg/l to lbs/cf

= 0.000062435.

The total flow generated by the combined sewer area is obtained from

$$QCOM = \sum_{i=1}^{r} (Q_c + DWFCMB)_i \tag{3.9}$$

where $i = 1, 2, 3, \ldots, r$, and

r = total number of hourly runoff occurrences during the year.

Finally, the mixed BOD concentration in the combined sewer, BOD_c (mg/l), is computed by the following expression:

$$BOD_c = \frac{BOD_t \cdot Q_c + BOD_f \cdot Q_d \cdot (A_c/A_t) + FF \cdot c_2}{Q_c + Q_d \cdot (A_c/A_t)} \tag{3.10}$$

where c_2 = factor to convert FF from lb/hr to cfs • mg/l = 4.45.

The first flush factor, FFLBS, is computed in the next section from available data sources. Further research is necessary to develop a mathematical relationship that predicts the first flush BOD load, independent of calibration to measured data, with a reasonable degree of accuracy.

3.8 Data Sources

Numerous agencies provided the information required to meet the substantial data needs for mathematical modeling of urban runoff, storage/treatment systems, and receiving water effects. All land use classifications, population density figures, area, curb lengths, etc., were obtained from data prepared by the American Public Works Association for specific use in STORM simulations. Precipitation records for 1968, collected at the Des Moines, Iowa Municipal Airport, were obtained from the National Weather Records Center at Asheville, North Carolina. Receiving water upstream flows, temperatures, BOD and DO levels were taken from the Davis and Borchardt report; however, supportive material which provided considerable insight on the characteristics of the Des Moines River was obtained from the State of Iowa Department of Environmental Quality.

The total hourly urban runoff (Q_t) and its BOD concentration (BOD_t) are obtained from the STORM simulation

of Des Moines, Iowa. BOD_s, Q_s, BOD_d, Q_d, BOD_c, and Q_w are computed from input data and the appropriate mixing equations presented earlier. BOD_w is obtained from the mathematical models of the storage/treatment process, discussed in Chapter V. The first flush factor is obtained from calibration of the digital computer model to measured data.

(1) From rainfall records,

TDWH = 6,993 DWH/yr

(2) From equation (2.10),

QCOM = 156,530,672 cf/yr

(3) From Davis and Borchardt (1974),

$\overline{BOD}_{CM}$ = 72 mg/l

(4) From digital computer simulation, disregarding in-sewer solids build-up, the average BOD concentration in the combined sewers = 61.92 mg/l

From the observed difference between items (3) and (4) above,

BOD difference = 10.08 mg/l

= 0.0006293 lbs/cf

$$\text{BOD load} = (0.0006293 \text{ lbs/cf})(156{,}530{,}672)$$

$$= 98{,}511.76 \text{ lbs/yr } (44{,}684 \text{ kg/yr})$$ that can be attributed to first flush effects

$$\text{FFLBS} = \frac{98{,}511.76 \text{ lbs/yr}}{6{,}993 \text{ DWH/yr}}$$

$$= 14.09 \text{ lbs BOD/DWH}$$

$$= 6.39 \text{ kg BOD/DWH}$$

This factor is then used in equation (3.7) to estimate the first flush BOD load, FF, during the first hour of runoff generated by each storm event. First flush effects account for 14 percent of the average BOD load in the combined sewer overflows.

CHAPTER IV

STOCHASTIC CHARACTERIZATION OF URBAN RUNOFF

4.1 Hydrologic Time Series

Before the unifying concept can be applied to wet weather and dry weather flows, it is important to examine and analyze the hydrologic time series. A time series is a sequence of values, arranged in order of their occurrence, which may be characterized by statistical properties. Data representing a purely random physical phenomenon, or even a physical process that contains a random element, cannot be described by an explicit mathematical relationship. A <u>hydrologic time series</u> may be considered to be the sum of two components: a <u>random</u> element and a <u>nonrandom</u> element. For a hydrologic time series, the occurrence of an event is not necessarily independent of all previous events (Dawdy and Matalas, 1964). In fact, when known deterministic components are removed (e.g., annual cycles) the dependence between hydrologic observations decreases with an increase in the time base. For example, the dependence between monthly observations is less than between daily observations (Dawdy and Matalas, 1964).

As stated previously, rainfall input to STORM is prepared as a sequence of consecutive hourly values (including zeros for no measurable precipitation). These inputs are used by STORM to generate the corresponding series of hourly urban runoff. The basic approach to define a wet weather event is to analyze the hydrologic time series and establish the minimum number of consecutive dry weather hours (DWH) that separates independent storm events. The independence of these storm events is not strictly climatological and is discussed later. The dry weather hours refer to periods during which no runoff was produced. Thus, depression storage and evaporation rates must be satisfied before any runoff is generated by STORM.

The precipitation time series for Des Moines, Iowa for the year of record 1968 is presented in Figure 4-1. The abscissa represents the 10-month period, in hours, from March 1 to December 30. An examination of the rainfall record provides considerable insight as to storm groupings, their intensity and duration, and frequency of occurrence. The broken line on the abscissa indicates dry weather periods at least nine hours in length. The annual precipitation total is 27.59 inches (701 mm), producing 10.28 inches (261 mm) of urban runoff as predicted by the urban hydrologic model.

4.2 Analysis of Time Series

The analysis of the hydrologic time series is performed at two levels: (1) an analytic approach,

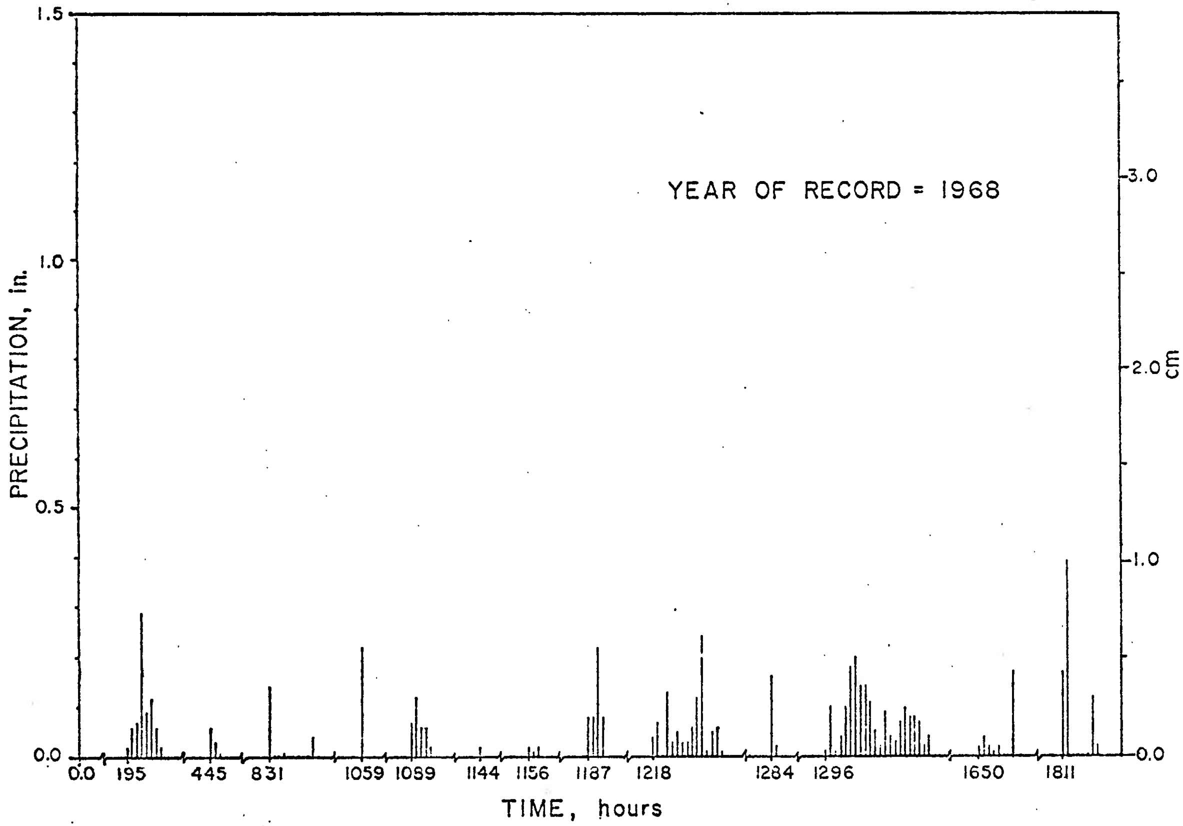


Figure 4-1. Point Rainfall for Des Moines, Iowa.

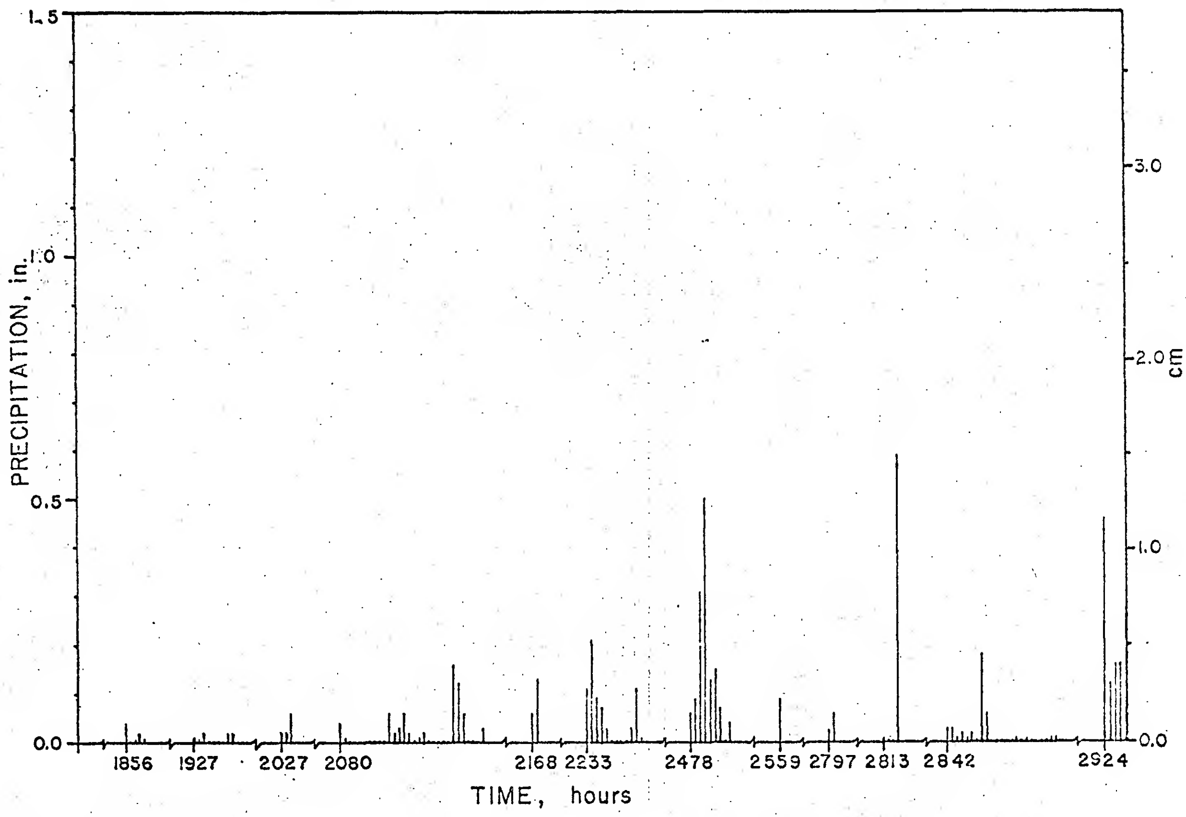


Figure 4-1 continued

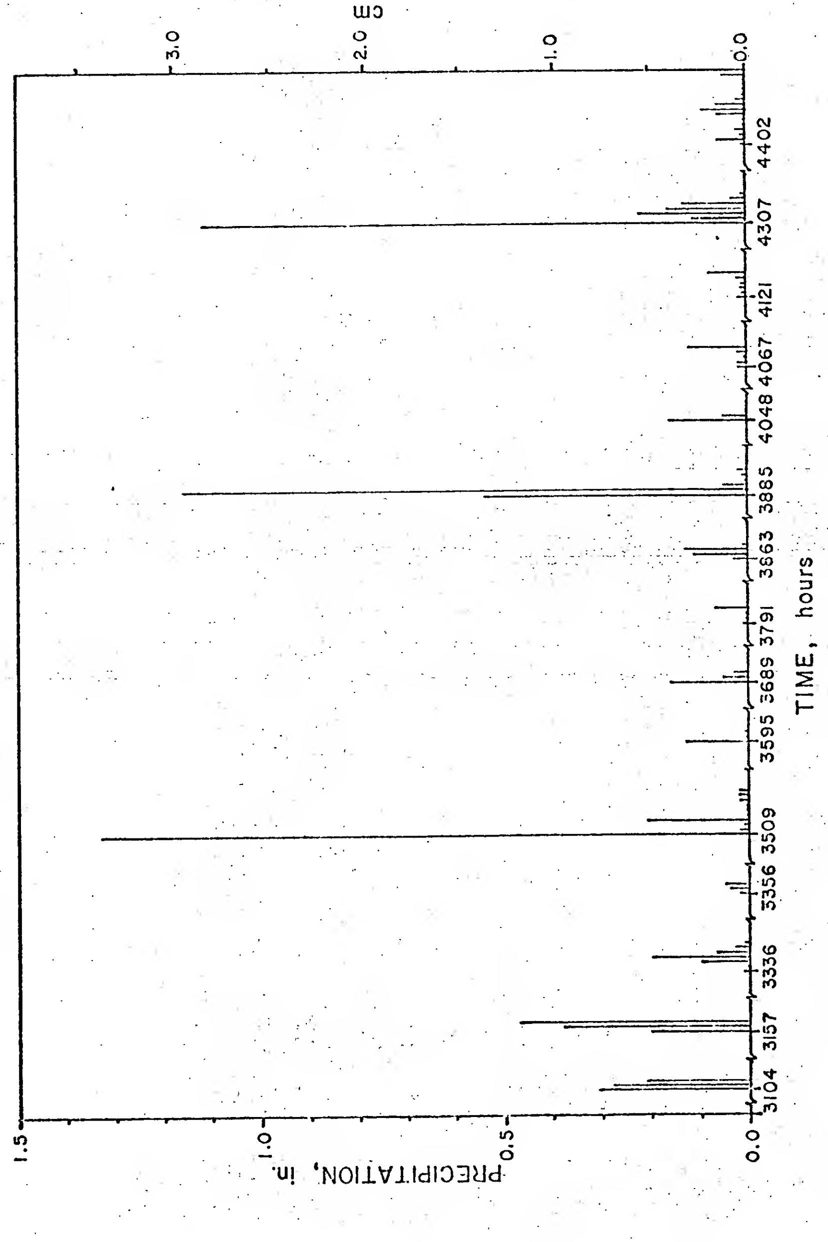


Figure 4-1 continued

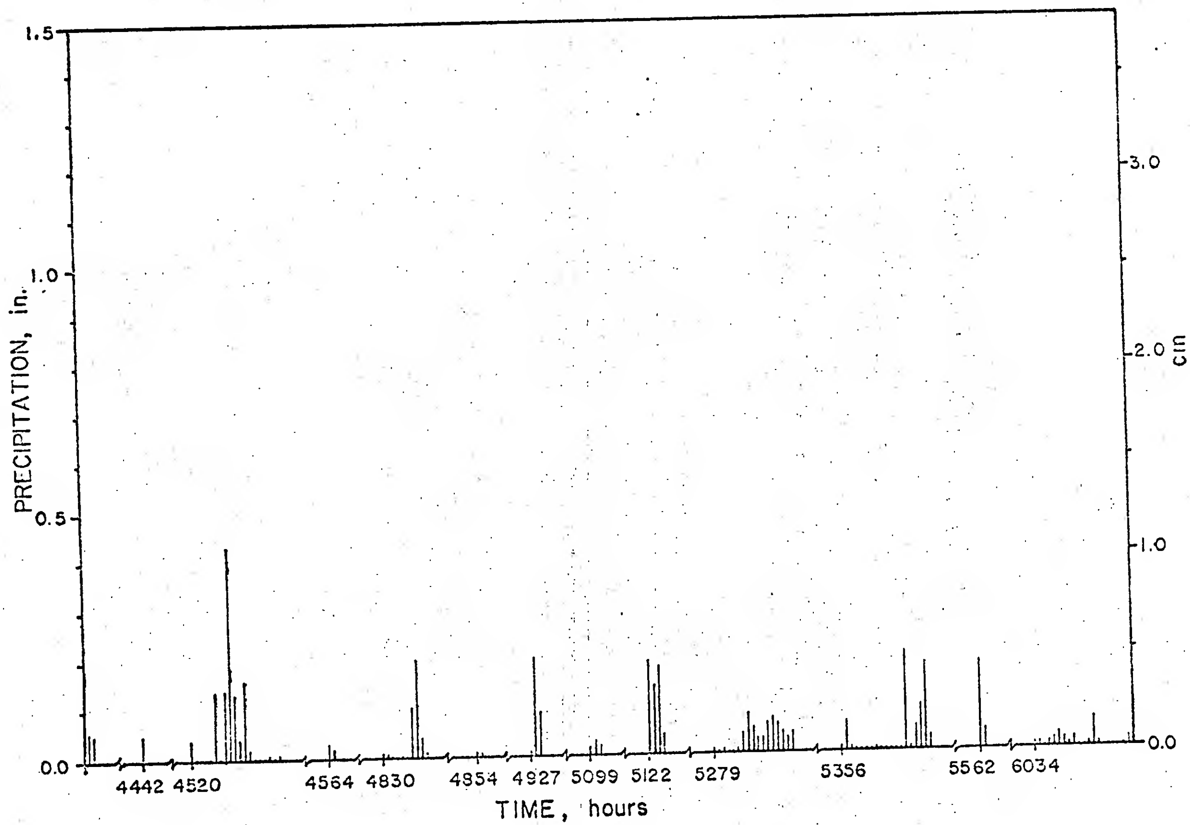


Figure 4-1 continued

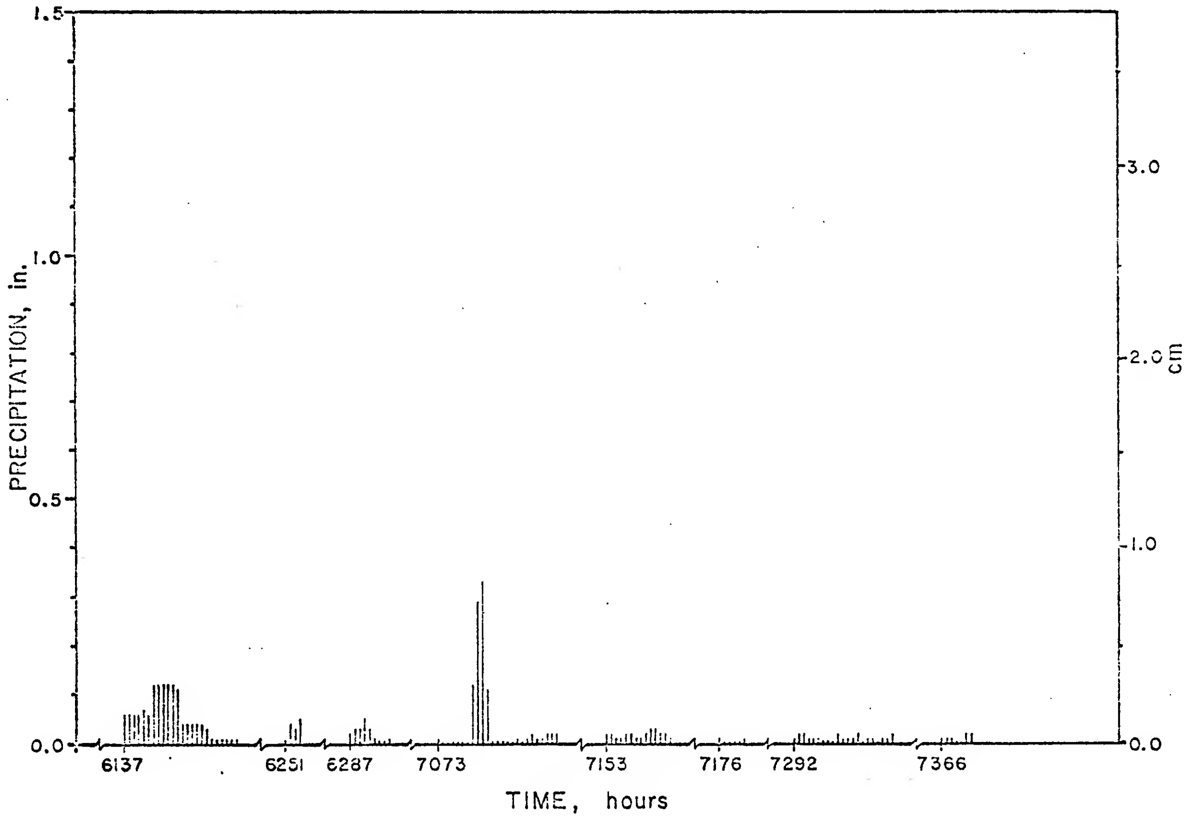


Figure 4-1 continued

autocorrelation, and (2) an approximate, graphic procedure. The results of the analytic approach are subjected to a parametric test of significance for serial dependence.

Two approaches may be used to compute sample autocorrelation coefficients: a circular series approach or an open series approach. In the circular series approach the end of a sample series is assumed to continue with its beginning (Yevjevich, 1972). For both continuous and discrete series in hydrology, the use of the circular series approach is not recommended because the first and the last part of the series may be independent, but corresponding adjacent parts may be highly dependent. The circular series approach may decrease the degree of dependence, thus introducing bias into the computation (Yevjevich, 1972). Thus, it is preferable to estimate the autocorrelation coefficients by an open series method.

$$r_I(k) = \frac{\sum_{i=1}^{n-k} x_i x_{i+k} - \frac{1}{n-k}\left[\sum_{i=1}^{n-k} x_i\right]\left[\sum_{i=k+1}^{n} x_i\right]}{\left[\sum_{i=1}^{n-k} x_i^2 - \frac{1}{n-k}\left(\sum_{i=1}^{n-k} x_i\right)^2\right]^{0.5}\left[\sum_{i=k+1}^{n} x_i^2 - \frac{1}{n-k}\left(\sum_{i=k+1}^{n} x_i\right)^2\right]^{0.5}} \tag{4.1}$$

where $r_I(k)$ = sample estimate of lag-k autocorrelation coefficient for hydrologic process I,

x_i = discrete data series (observations) of hydrologic process I, for i = 1, 2, ..., n,

n = total number of data points or observations, and

k = number of hourly lags.

In the open series approach decribed by equation (4.1), the first part of the series (n-k) long is used for x_i and the last part of the series (n-k) long is used for x_{i+k}. Equation (4.1) provides an unbiased and efficient estimate of the lag-k autocorrelation coefficient, which is also referred to as the <u>sample estimate</u> of the <u>lag-k serial correlation coefficient</u> (Fiering and Jackson, 1971).

A plot of the serial correlation coefficients, r(k), against the number of lags, k, is called a correlogram. The technique of autocorrelation analysis is essentially a study of the behavior of the correlogram of the process under investigation (Quimpo, 1968). The correlogram shape, or curve joining each point to the next, is henceforth referred to as the autocorrelation function. An analysis of the precipitation time series of Figure 4-1 results in the curve shown in Figure 4-2. At a lag of of zero hours, the correlation of the discrete open series is unity because this point on the curve represents the linear dependence of the data series on itself. The number of observations (including zero values) totals

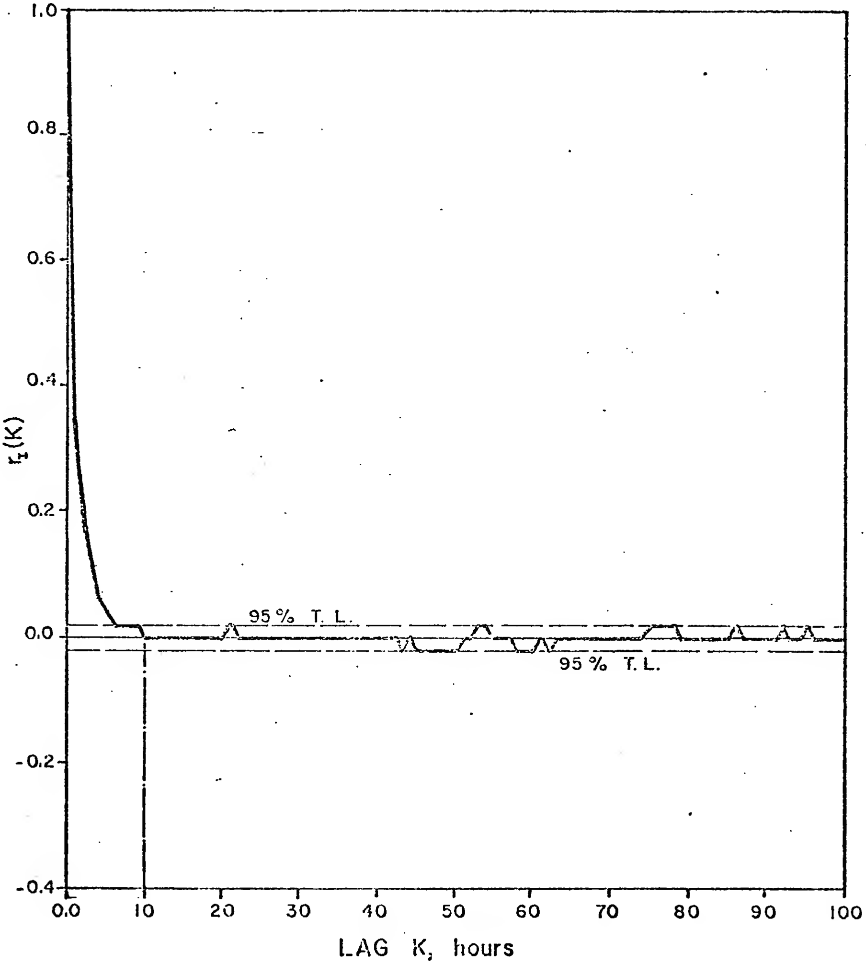


Figure 4-2. Lag k Autocorrelation Function of Des Moines, Iowa, Hourly Rainfall, 1968.

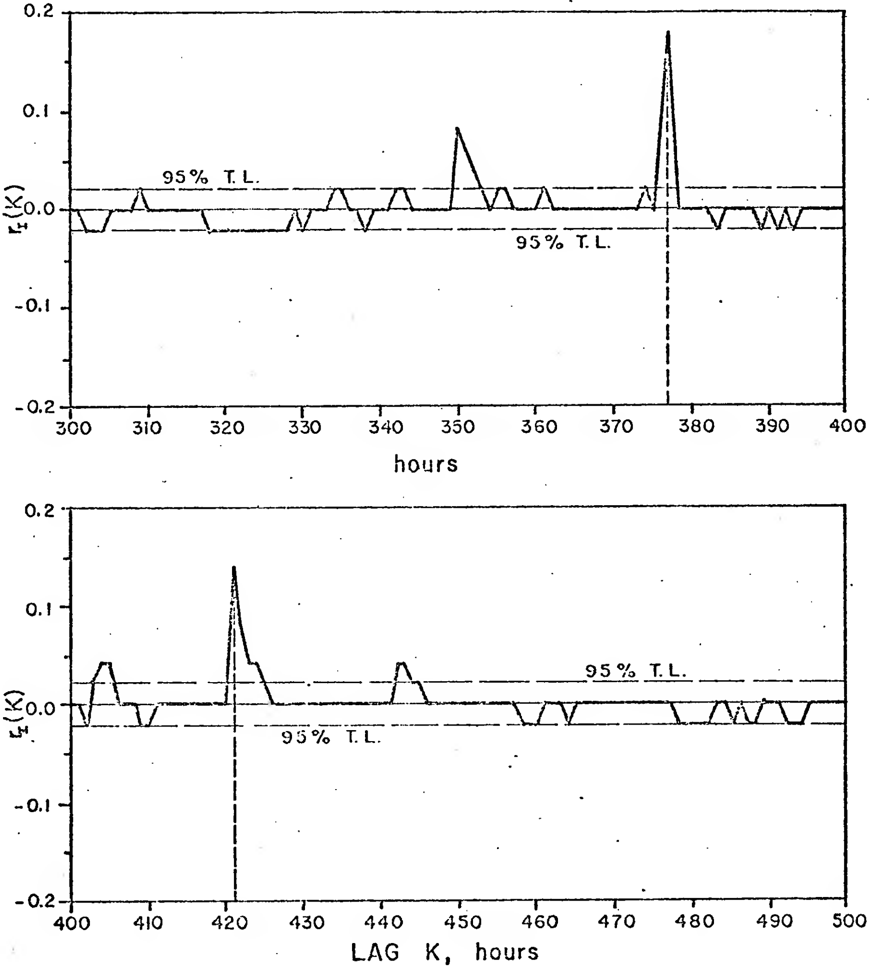


Figure 4-2 continued

7,372 consecutive values, and lags up to 720 hours were investigated. The first minimum of the autocorrelation function occurs at a lag of 10 hours, and the value of the function is also zero at this point. The physical interpretation is that periods without rainfall for at least 10 hours separate uncorrelated, and therefore independent, storm events.

Actually any point of the autocorrelation function which lies outside of the 95 percent tolerance limits indicated in Figure 4-2 suggests a significantly non-zero correlation between storm events at that particular time lag. The Des Moines rainfall record obviously exhibits nonrandom behavior at lags of 377 hours ($\sim$ 16 days) and 421 hours ($\sim$ 18 days) in particular. The tolerance limits for a normal random time series of N values, and an open series approach at a 95 percent probability level, are given by Yevjevich, 1972.

$$\mathrm{TL}\ (95\%) = \frac{-1 \pm 1.645\ \sqrt{N-k-1}}{N-k} \tag{4.2}$$

where TL (95%) = tolerance limits at a 95% probability level.

As the number of lags, k, increases, the tolerance limits diverge. However, the divergence is not noticeable for large N. Values of the autocorrelation function between

lags of 100 to 300 hours and 500 to 720 hours fell between the 95 percent tolerance limits and are not shown in Figure 4-2.

Similarly, autocorrelation analysis was performed on the sequence of hourly runoff values generated by STORM from the rainfall input. The lag-k serial correlation coefficients, $r_Q(k)$, are plotted against the number of lags in Figure 4-3. The analytic technique establishes that the minimum interevent time of consecutive DWH that separates independent runoff events is nine hours. Examination of Figure 4-3 reveals that the runoff time series is not purely random either. Linear dependence is observed at time lags of 377 hours ($\sim$ 16 days) and 436 hours ($\sim$ 18 days), as expected, because of the high correlation between rainfall and runoff processes.

Slight differences are observed between the correlograms of Figure 4-2 and 4-3. These are due to the fact that depression storage and evaporation rates must be satisfied before runoff is generated by STORM. Thus, the digital simulation of the runoff process by STORM acts as a filter and has a slight smoothing effect.

The graphic procedure requires the number of dry weather hours immediately preceding each hourly runoff occurrence. These values are determined directly by the chronological record provided by STORM of all the runoff events it generates from the input rainfall. If a hydrologic

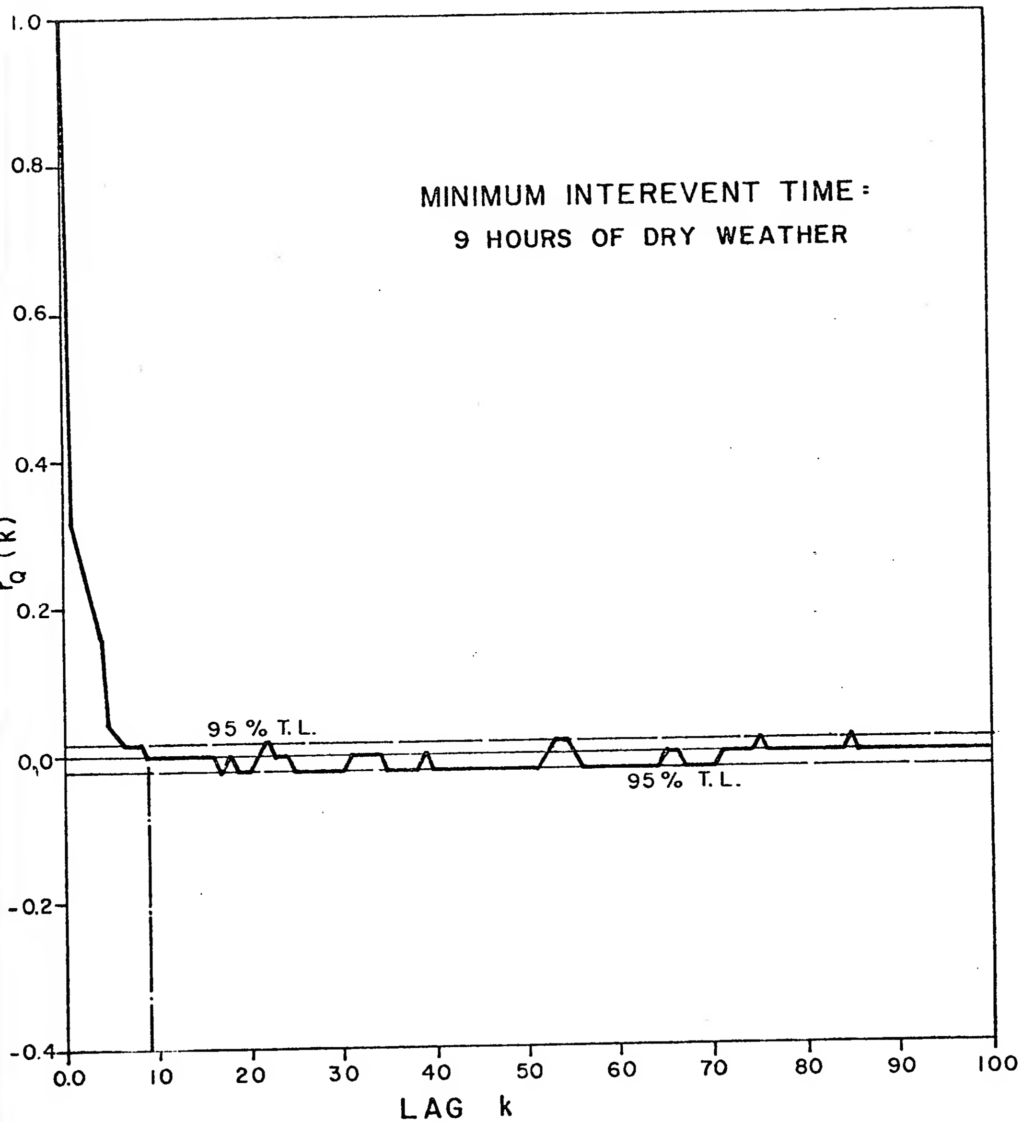


Figure 4-3. Autocorrelation Function of Hourly Urban Runoff for Des Moines, Iowa, 1968.

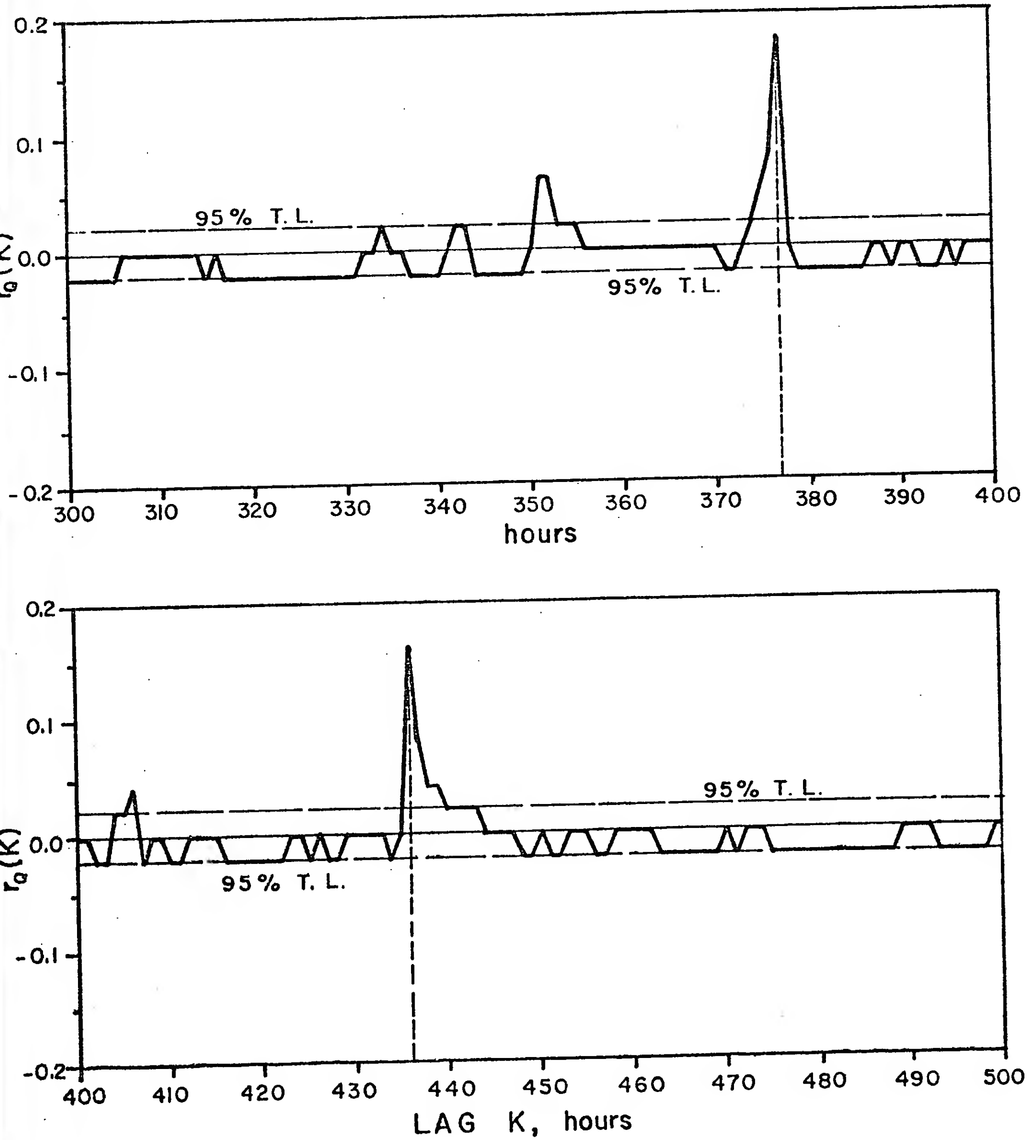


Figure 4-3 continued

model such as STORM is not available, a close approximation may be obtained by assuming that the same numbers of DWH precede the rainfall and runoff events. Thus, the information provided by the precipitation records or the rainfall time series (such as Figure 4-1) is sufficient. A plot of the number of wet weather events obtained by varying the minimum interevent time is shown in Figure 4-4. It is evident that a time value exists after which an increase in the minimum interevent time does not result in a correspondingly significant reduction in the number of storm events. By the graphic procedure, a period of eight consecutive DWH is obtained as the minimum interevent time. This result may be checked by constructing a divided difference table. From Table 4-1, the first minimum second difference occurs after the point of maximum curvature, at about 8 hours. These results indicate that the graphic procedure yields an approximate solution. Whenever possible, the analytic approach that defines a correlation should be applied to investigate the sequential properties of the hydrologic series.

4.3 Definition of an Event

Based on the above analyses, a wet weather event and its duration are defined in the mathematical model as follows:

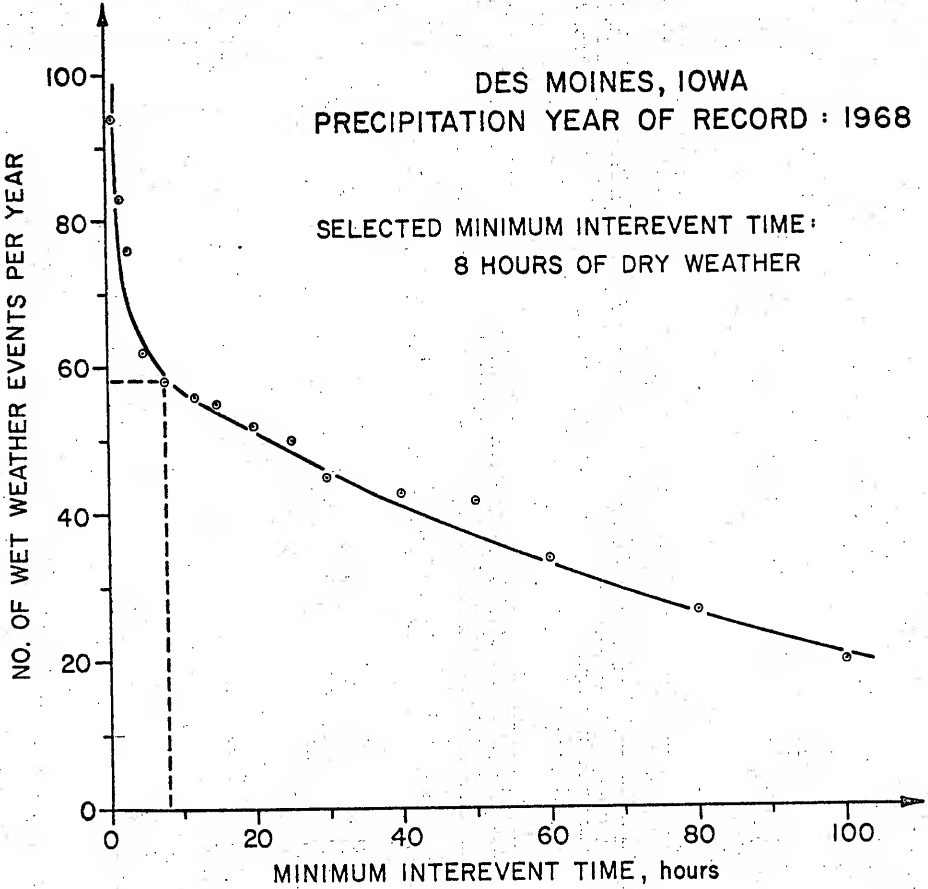


Figure 4-4. Definition of a Wet Weather Event for Des Moines by Graphic Procedure.

Table 4-1

Divided Difference Table for Determination of Point of Maximum Curvature

Minimum Interevent Time, hours	No. of Events	Divided Differences* Δ	Δ^2
2	82		
		6.00	
3	76		3.50
		13.00	
4	63		3.83
		1.50	
6	60		0.25
		0.50	
8	59		0.04
		0.75	
12	56		0.06
		0.33	
15	55		0.03
		0.60	
20	52		

*Maximum second difference, Δ^2, defines the point of maximum curvature.

1. Any runoff occurrence having nine or more DWH preceding it denotes the beginning of the event (see below).

2. The event continues as long as all of the subsequent runoff occurrences have a DWH value immediately preceding them equal to or less than eight hours.

3. The event runoff duration (in hours) is equal to the sum of all the runoff occurrences in (2).

4. The actual duration (in hours) must be determined by examining the date and hour of the first runoff value and the date and hour of occurrence of the last runoff value within the event.

When applied to the hydrologic time series of 1968 for Des Moines, Iowa, the above definition results in a total of 58 storm events encompassing 65 days and ranging from an event runoff duration of one hour to an actual event duration of 30 hours. The longest sequence of consecutive runoff hours is a wet weather event 24 hours in length, denoted hereafter as Wet Weather Event No. 52. The precipitation series generating this event begins at a time of 6137 hours on the abscissa of Figure 4-1, corresponding to a late December storm. The response of various storage/treatment systems to BOD inputs from this runoff event, in particular, and all wet weather events in general is presented in Chapter V.

4.4 Statistical Summaries of Deterministic Mechanisms

The mathematical relationships that explicitly describe system behavior, or response to time-varying

inputs, produce results which are specific to those inputs. It is therefore helpful to complement these techniques with statistical measures that allow the generalization of results (Benjamin and Cornell, 1970; Freund, 1971). Thus, a better perspective may be obtained. To illustrate, it is of interest to investigate the statistics of a water quality parameter entering the WWF storage/treatment facility shown in Figure 2-1, and also the statistics of that same water quality parameter after leaving the system. Regardless of the deterministic mechanisms used to describe the input to and output from that system, the relationships develped below will apply. In such a manner, the effectiveness of various system models may be studied.

For a sequence of observed (or computed) values, the sample mean is simply given by

$$\bar{x} = \frac{1}{n} \sum_{i=1}^{n} x_i \tag{4.3}$$

where $\bar{x}$ = unbiased estimate of the mean value of the water quality parameter,

x_i = data series of observed magnitudes of the water quality parameter, for i=1, 2, 3, ..., n, and

n = total number of observations.

The sample estimate of the variance is computed conveniently from

$$s^2 = \frac{1}{n-1} \sum_{i=1}^{n} X_i^2 - [\frac{n}{n-1}] (\overline{x})^2 \tag{4.4}$$

where s^2 = unbiased estimate of the variance of observed magnitudes of the water quality parameter.

The standard deviation is simply the square root of equation (4.4). The coefficient of variation is obtained from the standard deviation and the mean

$$c.v. = \frac{s}{\overline{X}} \tag{4.5}$$

where c.v. = sample coefficient of variation of the water quality parameter, and

s = standard deviation of observations of the water quality parameter from its computed mean.

Statistical measures of pollutant concentrations, unlike mass rates, require flow-weighting to obtain meaningful comparisons. A flow-weighted average is obtained as follows:

$$\overline{X}_w = \frac{\sum_{i=1}^{n} X_i \; Q_i}{\sum_{i=1}^{n} Q_i} \tag{4.6}$$

where $\overline{X}_w$ = unbiased estimate of the flow-weighted mean value of the pollutant concentration, and

Q_i = flows associated with observed magnitudes of the pollutant concentration, L^3/T.

Flow-weighted variances may be obtained similarly; however, a statistical approach based on recent work by DiToro (1975) is presented in Chapter VI which provides a technique for calculation of the coefficient of variation of concentration for the well-mixed constant volume equalization basin. Thus, variances are computed in Chapter V only for flow rates and mass rates, which provide sufficient information on the performance of the storage/treatment devices.

CHAPTER V

MATHEMATICAL APPROACH TO STORAGE/TREATMENT RESPONSE TO VARIABLE FORCING FUNCTIONS

5.1 System Models and the Unifying Concept

The applicability of the one-dimensional version of the conservation of mass equation to various components of the physical system has been discussed in Chapter II. Three mathematical models of the storage/treatment unit are investigated throughout this chapter:

- -the well-mixed constant volume model,
- -the well-mixed variable volume model, and
- -the dispersive variable volume model.

These mass transport models are derived from simplification of the same differential equation, equation (2.1). The methodology presented for completely mixed systems may also be used to approximate, for example, pollutant transport through lakes and bays.

The storage/treatment unit has become an essential element of decision perspectives on urban stormwater control. As shown in Figure 3-4, it has been assumed for modeling purposes that the urban runoff generated by precipitation over areas served by separate or combined

sewers is processed through a single, central storage/treatment facility prior to release into receiving waters. As a subsystem, it is acted upon by external forces and as a result produces certain effects. Its inputs are referred to as forcing functions and output as responses. The response of the system to a given forcing function depends on the type of mathematical model chosen to describe its properties and behavior.

Forcing functions may be continuous functions of time, or defined only at discrete intervals of time. A discrete function may be treated as piecewise continuous if the value of the function is constant for each time interval where it is defined. Thus, the value of a discrete forcing function may change abruptly from one time interval to the next. In that sense, a discrete forcing function may be regarded as a step function input. Regardless of the form of the input function, the mathematical models define the response of the system as a continuous function of time. To compare effluent quality and quantity among the three mathematical models of the storage/treatment system, a common parameter was selected: the residence time of inflows. It was desired to select a broad range of residence times so that generalizations on system behavior could be made. In sewage treatment works practice, the size of sedimentation tanks is calculated by the design overflow rate. It represents the unit

volume of flow per unit of time divided by the unit of tank surface area (American Society of Civil Engineers, 1959). The selection of the overflow rate is a matter of engineering judgment and experience, but usually the recommendations on the Ten-State Standards are followed (Great Lakes-Upper Mississippi River Board of State Sanitary Engineers, 1971). These surface-loading or surface-settling rates range from 600 gpd/ft^2 (211 l/day/m^2) to 1000 gpd/ft^2 (352 l/day/m^2). In effect, this overflow rate is the average upflow velocity as confirmed by examining the units in which it is expressed. Based on average design flow and tank depths, overflow rates in current use result in nominal detention periods of 2 to 2.5 hours (Metcalf & Eddy, Inc., 1972).

In industrial water pollution control practice, stabilization basins are the most common method of organic waste treatment where sufficient land area is available (Eckenfelder, 1966). In particular, well-mixed aerated lagoons with depths of 6 to 15 feet (1.8 to 4.6 m) are designed to provide 2 to 10 days of detention (Eckenfelder, 1966). However, industrial flows in an urban area are much smaller than stormwater flows. To illustrate, the combined industrial and municipal wastewater flow for Des Moines, Iowa, is approximately 55 cfs (1.6 cu m/sec). The average urban stormwater flow (including combined sewer overflow) is approximately 1600 cfs (45 cu m/sec)

for a precipitation year close to the national average. This represents a 1:29 ratio. Of course, this ratio will be highly variable depending on population densities, regional climatology, and regional industrialization. The illustration underscores the fact that the surface area required to provide detention times on the order of days is not likely to be available in Standard Metropolitan Statistical Areas (SMSA's) in terms of both space and cost of providing it. Thus, four residence times were selected for investigation for each system model.

-a 2-hour detention time,

-a 6-hour detention time,

-a 12-hour detention time, and

-a 24-hour detention time.

In addition, a 48-hour detention time was studied using the variable volume model to represent the storage/treatment system, and results are presented for a single storm event to confirm that increased flow equalization occurs.

Although applications of the mathematical models are made to Des Moines, Iowa, the results are not site limited. As mentioned in Chapter III, Des Moines and the Des Moines River represent an urban area-receiving water configuration common throughout the country. The data base is typical of that available to engineers involved in studies of areawide waste management. Generalizations are discussed in the summary at the end of this chapter.

5.2 Well-Mixed Constant Volume Model

Continuous Forcing Functions

If the storage/treatment facility for urban run-off is modeled as a well-mixed constant volume tank, the system may be represented as shown in Figure 5-1. The volumetric rates of flow into and out of the tank are equal and constant. However, the concentration forcing function and response are continuous functions of time. The pollutant mass in the tank is assumed to undergo a first-order reaction. From a mass balance of the dissolved material across the tank, the governing equation is given by

$$V \frac{dc_2}{dt} = Q\, c_1(t) - Q\, c_2(t) - K\, c_2(t)\, V \qquad (5.1)$$

where V = volume of tank, L^3.

$c_2(t)$ = concentration of material in the tank and outflow, as a continuous function of time, M/L^3,

Q = volumetric flow into and out of tank, L^3/T,

$c_1(t)$ = concentration forcing function of material in inflow, M/L^3,

K = first-order decay rate constant, $1/T$, and

t = time, T.

Equation (5.1) is the mathematical expression that describes the response of the system, $c_2(t)$, to a

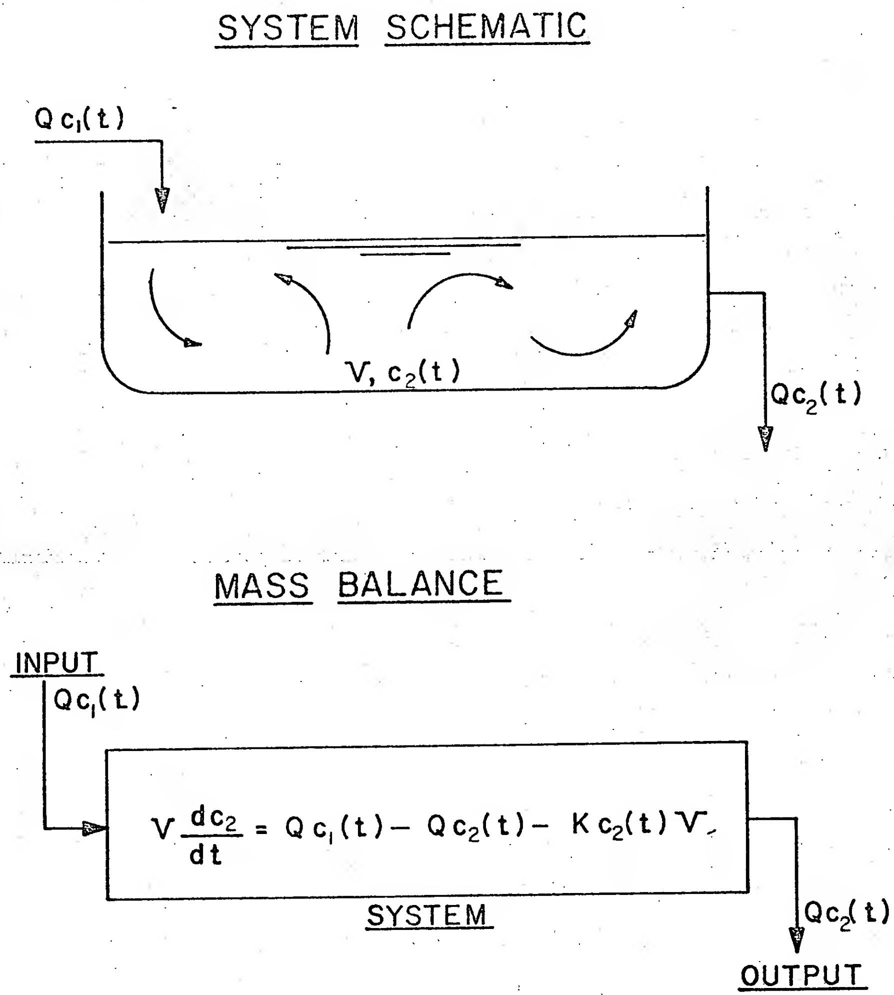


Figure 5-1. Well-Mixed Constant Volume Model.

time-varying forcing function of concentration, $c_1(t)$. Since this model is a linear, first-order ordinary differential equation with constant coefficients, analytic solutions are not difficult to obtain. Rearranging equation (5.1) to a more convenient form,

$$\frac{dc_2}{dt} + \left[\frac{Q}{V} + K\right] c_2(t) = \frac{Q}{V}\, c_1(t) \tag{5.2}$$

and letting

$$a = \left[\frac{Q}{V} + K\right],$$

equation (5.2) is further simplified to

$$\frac{dc_2}{dt} + a\, c_2(t) = \frac{Q}{V}\, c_1(t) \tag{5.3}$$

The inverse of a is a measure of the time required by the system to respond to a change in the input concentration, $c_1(t)$. The general solution to equation (5.3) is obtained by the method of integrating factors (Ross, 1964) and is

$$c_2(t) = e^{-\int a\,dt}\left[\int e^{\int a\,dt}\, \frac{Q}{V}\, c_1(t)\,dt + \beta\right] \tag{5.4}$$

where β = constant of integration.

By examining the initial condition of the system, it is

observed that at $t = 0$, $c_2(t=0) = \beta$, and equation (5.4) is reduced to the form

$$c_2(t) = \frac{Q}{V} e^{-at} \int_0^t c_1(t)e^{at}dt + c_2(t=0)e^{-at} \tag{5.5}$$

At this stage, complete solution of equation (5.5) is dependent on the form of the concentration forcing function, $c_1(t)$. This continuous function of time may be constant, linear, exponential, or of other types. Solutions to various common forcing functions, for constant volume and constant flow rate, are presented in Table 5-1, as adapted from Rich (1973). It is important to emphasize that these solutions pertain to continuous-flow systems. Thus, the forcing function and the response have been expressed as continuous functions of time: $c_1(t)$ and $c_2(t)$, respectively. The reference to function of time, t, has been deleted in the derivative terms for notational simplicity.

The above analysis may be extended to a storage facility maintaining constant volume, but subjected to time-varying forcing functions of both concentration and flow. Referring to Figure 5-1, the flow rate Q is now a continuous function of time, Q(t), and the governing equation is given by

$$V \frac{dc_2}{dt} = Q(t) \left[c_1(t) - c_2(t)\right] - K\, c_2(t)V \tag{5.6}$$

Table 5-1

Solutions to Equation (5.5) for Various Forcing Functions of Concentration*
(Rich, 1973)

Type of Input	Form of $c_1(t)$	Output Concentration $c_2(t)$
Constant	c_1	$\frac{c_1}{a}\,\frac{Q}{V}\,(1-e^{-at})+c_2(t=0)e^{-at}$
Linear	$c_1 \pm bt$	$\frac{c_1}{a}\,\frac{Q}{V}\,(1-e^{-at}) \pm \frac{b}{a^2}\,\frac{Q}{V}\,(1-e^{-at}-at) + c_2\,(t=0)e^{-at}$
Exponential	$c_1\,e^{\pm bt}$	$\frac{c_1}{a \pm b}\,\frac{Q}{V}\,(e^{\pm bt} - e^{-at}) + c_2\,(t=0)e^{-at}$

* b = arbitrary constant

$a = \frac{Q}{V} + K$, where K = first-order decay rate constant.

Again, by rearranging into a more convenient form,

$$\frac{dc_2}{dt} + \left[\frac{Q(t)}{V} + K\right] c_2(t) = \frac{Q(t)}{V} \quad c_1(t) \tag{5.7}$$

and letting

$$a(t) = \frac{Q(t)}{V} + K, \text{ then}$$

$$\frac{dc_2}{dt} + a(t)\ c_2(t) = \frac{Q(t)\ c_1(t)}{V} \tag{5.8}$$

The general solution to equation (5.8), a linear, first-order, ordinary differential equation with variable co-efficients is also obtained by the method of integrating factors (Ross, 1964).

$$c_2(t) = e^{-\int_0^t a(t)dt} \int_0^t e^{\int_0^t a(t)dt} \frac{Q(t)\ c_1(t)\ dt}{V}$$

$$+ c_2(t{=}0)\ e^{-\int_0^t a(t)dt} \tag{5.9}$$

where $e^{\int_0^t a(t)dt}$ = integrating factor.

The complete integration of equation (5.9) is dependent on the form of the two forcing functions, $Q(t)$ and $c_1(t)$.

Solutions to various types of both forcing functions are presented in Table 5-2. Integration has been accomplished as far as analytically possible. An alternative method is expansion into series and integration of a number of terms until the approximation is considered adequate. However, it is simpler to discretize the input to obtain a numerical solution on the digital computer.

Discrete Inputs

Observed data provide information about physical phenomena and may be broadly classified as being either deterministic or nondeterministic (random). In practice, no physical data are truly deterministic nor truly random (Bendat and Piersol, 1971), as discussed previously. The forcing functions of flow rate and concentration presented in Tables 5-1 and 5-2 are explicit mathematical relationships. However, information systems about hydrologic phenomena and water quality processes are usually designed to deliver parameter measurements at discrete intervals for a finite period of time. Typical configurations of these inputs are shown in Figure 5-2.

The response of the storage/treatment system to discrete inputs may be derived from the analysis performed for continuous forcing functions. From equation (5.9),

Table 5-2

Analytic Solutions to Equation (5.9) for Various Forcing Functions of Concentration and Flow Rate*

Form of $c_1(t)$	Form of $Q_1(t)$	Output Concentration $c_2(t)$
Linear $c_1 + bt$ where b, γ may be either positive or negative	Exponential $Qe^{+\gamma t}$	$\exp\left[- \frac{Q}{\gamma V} (e^{+\gamma t}-1) - Kt\right] \cdot$ $\Big[\exp (- \frac{Q}{\gamma V} [(c_1 + bt) \exp (\frac{Q}{\gamma V} e^{+\gamma t}$ $+ Kt) - c_1 \exp (\frac{Q}{\gamma V}) -$ $(c_1 K+b) \int_0^t \exp (\frac{Q}{\gamma V} e^{+\gamma t} + Kt) dt$ $- \int_0^t \exp (\frac{Q}{\gamma V} e^{+\gamma t} + kt) tdt$ $+ c_2 (t=0)\Big]$
Exponential $c_1 e + bt$	Exponential $Qe^{+\gamma t}$	$\exp\left[- \frac{Q}{\gamma V} (e^{+\gamma t} - 1) - Kt\right] \cdot$ $\Big[c_1 \exp (- \frac{Q}{\gamma V}) \Big[\exp (\frac{Q}{\gamma V} e^{+\gamma t} +$ $Kt + bt) - \exp (\frac{Q}{\gamma V}) -$ $(K + b) \int_0^t \exp [\frac{Q}{\gamma V} e^{+\gamma t}] \cdot$ $\exp [(K + b) t] dt\Big] + c_2 (t=0)\Big]$

Table 5-2 continued

Form of $c_1(t)$	Form of $Q_1(t)$	
Exponential $c_1\ e^{+bt}$ where α may be positive or negative	Linear $Q = \alpha t$	$\exp\left[-\frac{Q}{V}t + \frac{\alpha}{2V}t^2 + Kt\right] \cdot \left[c_1 \exp\left(\frac{Q}{V}t + \frac{\alpha}{2V}t^2 + Kt + bt\right) - c_1 - c_1(K+b)\int_0^t \exp\left(\frac{Q}{V} + \frac{\alpha}{2V}t^2 + Kt + bt\right) dt^{**} + c_2\ (t=0)\right]$

*Constant Volume Storage/Treatment System.

**Note that when α is negative the integral is in the form of an error function, allowing simplification.

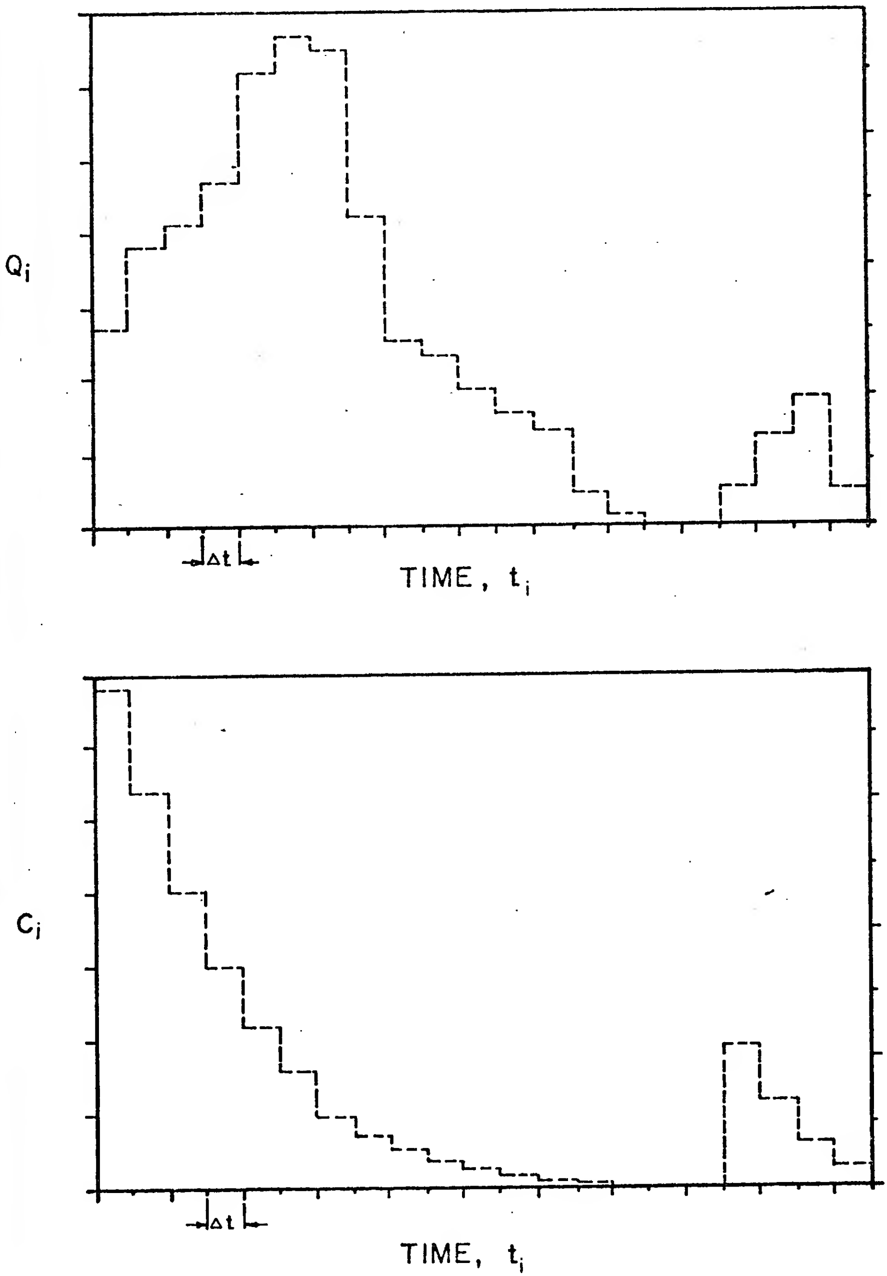


Figure 5-2. Discrete Inputs of Flow Rate and Concentration at Equal Time Intervals.

$$c_2(t_i + \Delta t) = \exp\left[-\int_{t_i}^{t_i+\Delta t} \left(\frac{Q_i}{V} + K\right) dt\right] \cdot$$

$$\left[\int_{t_i}^{t_i+\Delta t} \exp\left[\int_{t_i}^{t_i+\Delta t} \left(\frac{Q_i}{V} + K\right) dt\right] \frac{Q_i c_i}{V} dt\right.$$

$$\left. + c_2(t_i)\right] \qquad (5.10)$$

where Q_i = value of input flow rate from t_i to $(t_i+\Delta t)$, L^3/T,

c_i = value of input concentration from t_i to $(t_i+\Delta t)$, M/L^3,

Δt = length of time interval, say, 1 hour, and

t_i = the beginning of the time interval for which the system response is being evaluated, T.

For computational convenience, the forcing function may be treated as a step function input. Thus, initial conditions are assumed to be defined at time t_i by $c_2(t_i)$, the solution at the end of the previous time step. This allows simplification of equation (5.10) to

$$c_2(t_i + \Delta t) = \exp\left[-\int_0^{\Delta t} \left(\frac{Q_i}{V} + K\right) dt\right] \cdot$$

$$\int_0^{\Delta t} \exp\left[-\left(\frac{Q_i}{V} + K\right) t\right] dt +$$

(continued)

$$c_2(t_i) \exp\left[-\int_0^{\Delta t}\left(\frac{Q_i}{V} + K\right) dt\right] \qquad (5.11)$$

Equation (5.11) integrates to

$$c_2(t_i + \Delta t) = \exp\left[-\left(\frac{Q_i}{V} + K\right)\Delta t\right] \cdot$$

$$\left[\frac{Q_i c_i}{Q_i + KV}\left(\exp\left[\frac{Q_i \Delta t}{V} + K\Delta t\right] - 1\right) + c_2(t_i)\right] \qquad (5.12)$$

The average concentration for each time step may be obtained by integrating equation (5.12) over the length of the time step, as follows:

$$\bar{c}_2(t_i + \frac{\Delta t}{2}) = \frac{1}{\Delta t}\int_0^{\Delta t} c_2(\tau)\, d\tau \qquad (5.13)$$

where τ = dummy variable of integration.

Substituting equation (5.12) for $c_2(\tau)$, and noting that $c_2(t_i)$ is an initial value,

$$\bar{c}_2(t_i + \frac{\Delta t}{2}) = \frac{1}{\Delta t}\int_0^{\Delta t} \frac{Q_i c_i}{Q_i + KV}\left[1 - \exp\left(-\frac{Q_i \tau}{V} - K\tau\right)\right] d\tau$$

$$+ \frac{1}{\Delta t} c_2(t_i) \int_0^{\Delta t} \exp\left(-\frac{Q_i \tau}{V} - K\tau\right) d\tau \qquad (5.14)$$

and simplifying further,

$$\bar{c}_2(t_i + \frac{\Delta t}{2}) = \frac{Q_i c_i}{Q_i + KV} - \frac{Q_i c_i}{Q_i + KV} \cdot \frac{1}{\Delta t} \int_0^{\Delta t} \exp\left(-\frac{Q_i \tau}{V} - K\tau\right) d\tau + \frac{1}{\Delta t} c_2(t_i) \int_0^{\Delta t} \exp\left(-\frac{Q_i \tau}{V} - K\tau\right) d\tau \tag{5.15}$$

Finally, integrating equation (5.15), the solution is obtained.

$$\bar{c}_2(t_i + \frac{\Delta t}{2}) = \frac{Q_i c_i}{Q_i + KV} - \left[\frac{Q_i c_i}{Q_i + KV} - c_2(t_i)\right] \cdot \frac{1}{\Delta t}\left[1 - \exp\left[-\left(\frac{Q_i}{V} + K\right)\Delta t\right]\right] \frac{V}{Q_i + KV} \tag{5.16}$$

where $c_2(t_i)$ = concentration value obtained by equation (5.12) for the previous time step, M/L^3, and

$\bar{c}_2(t_i + \frac{\Delta t}{2})$ = time-averaged concentration for each time step, M/L^3.

When values of equation (5.16) are plotted with respect to time, the average value for each interval of the step function is assumed to occur at the mid-point of the time step. Thus, the notation $(t_i + \frac{\Delta t}{2})$ has been used.

Since the input and output mass rates are also of interest, these are computed from the known concentration and flow rates. The discrete mass rate input is

given by

$$W_i = Q_i \cdot c_i \tag{5.17}$$

where W_i = value of input mass rate from t_i to $(t_i + \Delta t)$, M/T.

Because inflow equals outflow during each time step, even though the flow-through rate may change from one time interval to another, the response of the storage/treatment system to the mass rate input is given by

$$\overline{w}_2(t_i + \frac{\Delta t}{2}) = Q_i \cdot \overline{c}_2(t_i + \frac{\Delta t}{2}) \tag{5.18}$$

where $\overline{w}_2(t_i + \frac{\Delta t}{2})$ = time-averaged mass rate output for each time step, M/T.

Again, when values of equation (5.18) are plotted with respect to time, the average value for each interval of the step function is assumed to occur at the mid-point of the time step. If the flow rate and concentration in equations (5.17) and (5.18) are not expressed in compatible units, an appropriate conversion factor is used.

Organic Decay and Sedimentation Rates

Colston (1974) investigated the characteristics of urban land runoff from a 1.67-square-mile (4.33 sq. kms) watershed in Durham, North Carolina. Thirty-six

separate storms were sampled; peak discharges varied from 2.25 to 1740 cfs, and the number of dry days preceding each storm varied from 0.5 to 34. The standard BOD test was performed at several dilutions (from 0.5 percent to 5.0 percent of the stormwater concentration). The COD test was performed at 100 percent strength of the stormwater concentration. Both the BOD and COD organic decay rates, K_d, were determined by the method of moments. This procedure is one of four generally accepted methods for computing the reaction rate of the BOD curve (Nemerow, 1974). There are numerous problems associated with planning BOD experiments (Berthouex and Hunter, 1971) and obtaining precise parameter estimates. Colston judged the standard BOD test as inappropriate for determining the ultimate oxygen demand of urban runoff.

However, it is significant that the COD K_d rates varied from 0.06 to 0.20 per day, while those determined by the conventional BOD test varied from 0.08 to 0.27 per day. From these studies, Colston concluded that the COD K_d exertion rate is equivalent to that obtained by a standard BOD test. The average COD decay rate, at 100 percent sample strength, was reported to be 0.13 per day (base e).

Based on results reported by Colston, the organic decay rate for BOD prior to mixing with the receiving waters is assumed to be

$$K_d = 0.005 \text{ hour}^{-1} \tag{5.19}$$

Thus, while stormwater is held in storage, treatment will occur naturally at the above deoxygenation rate. In addition, even in relatively well-mixed storage/treatment units some degree of sedimentation will occur. Figure 5-3 shows the BOD removal efficiency of primary sedimentation tanks treating municipal sewage (modified after Fair, Geyer, and Okun, 1968). The exponential relationship shown in Figure 5-3 is accurate for BOD influent concentrations ranging from 100 to 300 mg/l, and for relatively long rectangular tanks with a flow regime approaching a plug flow pattern.

In the calculation of the BOD washoff rate from an urban watershed, STORM includes contributions of 10 percent of the suspended solids washoff rate and 2 percent of the settleable solids washoff rate on an hourly basis (Hydrologic Engineering Center, 1975). The washoff rate of suspended solids from an urban watershed may be, depending on land use and soil cover, quite large. Thus, it is possible that a large percentage of the predicted BOD load in urban stormwater runoff is derived from sediment. In order to simulate the treatment achieved by settling in a relatively well-mixed storage facility, a maximum BOD removal of 10 percent is imposed on the exponential relationship shown in Figure 5-3. Then, an

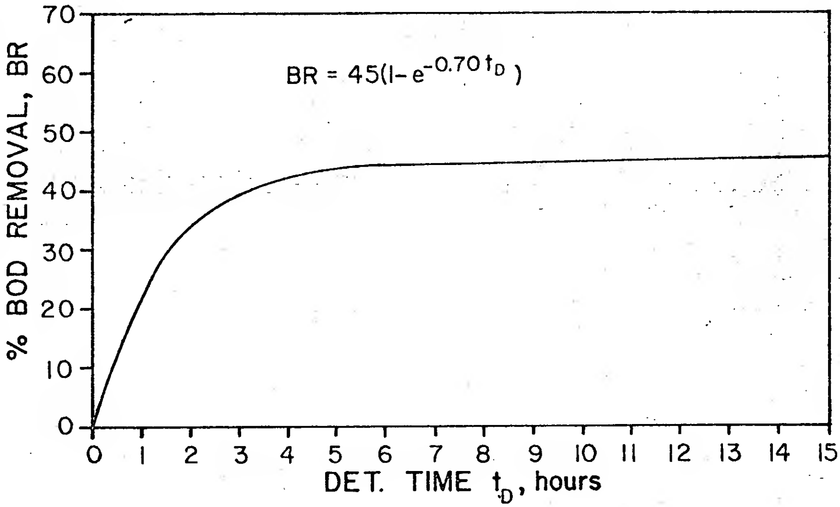


Figure 5-3. BOD Removal from Sewage in Primary Sedimentation Tanks. (Adapted from Fair, Geyer, and Okun, 1968.)

equivalent sedimentation rate constant, K_s, is derived as follows:

$$1 - \exp(-K_s \, \Delta t) = 0.10 \, [1 - \exp(-0.70 \, t_D)] \qquad (5.20)$$

where Δt = length of time step, T, and

t_D = detention time, T.

Solving for K_s, the following expression is obtained:

$$K_s = -\frac{1}{\Delta t} \, \ell n[0.90 + 0.10 \exp(-0.70 \, t_D)] \qquad (5.21)$$

where, for constant volume

$$t_D = \frac{V}{Q_i} \qquad (5.22)$$

The organic decay rate and the sedimentation rate are combined into an overall decay rate constant

$$K = K_d + K_s \qquad (5.23)$$

which is subsequently used in equations (5.12) and (5.16) to obtain the output concentration. When the time step length and the detention time are expressed in hours, the units of K are hours^{-1}.

A storage/treatment unit for wet weather flows is essentially a large equalization basin. Its function

is to dampen extreme fluctuations of concentration and mass flow rate by providing a well-mixed volume with a sufficiently long detention time. Equalization is provided by both the volumetric mixing of the influent with the wastewater already present in the facility and the decay action in the system. It is clear from equation (5.22) that the nominal detention time of the basin will vary according to the influent fluid flow rate, Q_i. An average of the time the fluid is held in the system may be obtained by

$$\bar{t}_D = \frac{1}{n} \sum_{i=1}^{n} \frac{V}{Q_i} \tag{5.24}$$

where $\bar{t}_D$ = average retention time, T,

V = constant volume of the storage/treatment system, L^3,

Q_i = variable influent fluid flow rate, L^3/T,

i = 1, 2, 3, ..., n, and

n = total number of time intervals of equal duration.

It is important to note that an erroneous average detention time would be obtained by dividing the basin volume by the computed average flow rate.

Interevent Decay Process

It is important not to overlook the fact that the wastewater in the storage/treatment system, between storm events, is also undergoing a first-order decay process. From knowledge of the number of DWH between storm events, and an accountability of the mixed basin concentration during the last hour of runoff of each storm event, the following relationship may be developed:

$$c_0(I) = c_2(I-1) \exp[-K_d \, DWT(I)] \tag{5.25}$$

where $c_0(I)$ = concentration of wastewater in the storage/treatment system at $t = 0$, for storm event I, M/L^3,

$c_2(I-1)$ = concentration of wastewater in the system during the last hour of runoff of the previous storm event, M/L^3, and

$DWT(I)$ = number of DWH preceding storm event I.

It has been assumed that the first-order decay process during interevent times is represented adequately by the organic decay rate. Thus, any sedimentation occurring in the basin during dry weather periods between storms is assumed to have a negligible effect on the BOD concentration of the quiescent wastewater.

Model Application

A description of the study area has been presented in Chapter III. An analysis of the hydrologic time series and a statistical definition of a wet weather event are discussed in Chapter IV. In brief review, an urban area consisting of 45,000 acres (18,212 ha) of separate sewers and 4000 acres (1619 ha) of combined sewers must provide storage/treatment for urban runoff generated by a total annual precipitation of 27.59 inches (701 mm). The total urban runoff predicted by the hydrologic simulation model is 10.28 inches (261 mm), but hourly runoff occurrences are grouped into 58 storm events by defining a minimum interevent time of 9 dry weather hours. Input and output hydrographs and pollutographs are presented for one storm event, but frequency analyses include all runoff occurrences during the year.

Discrete forcing functions of flow, BOD concentration and BOD mass rate for Wet Weather Event No. 52 are shown, respectively, in Figures 5-4, 5-5, and 5-6. From digital computer simulation using equation (5.24), the volumes (for all events) required to maintain average detention times of 2, 6, 12 and 24 hours in the storage/treatment facility are presented in Table 5-3. As stated previously, it is assumed that the entire urban runoff

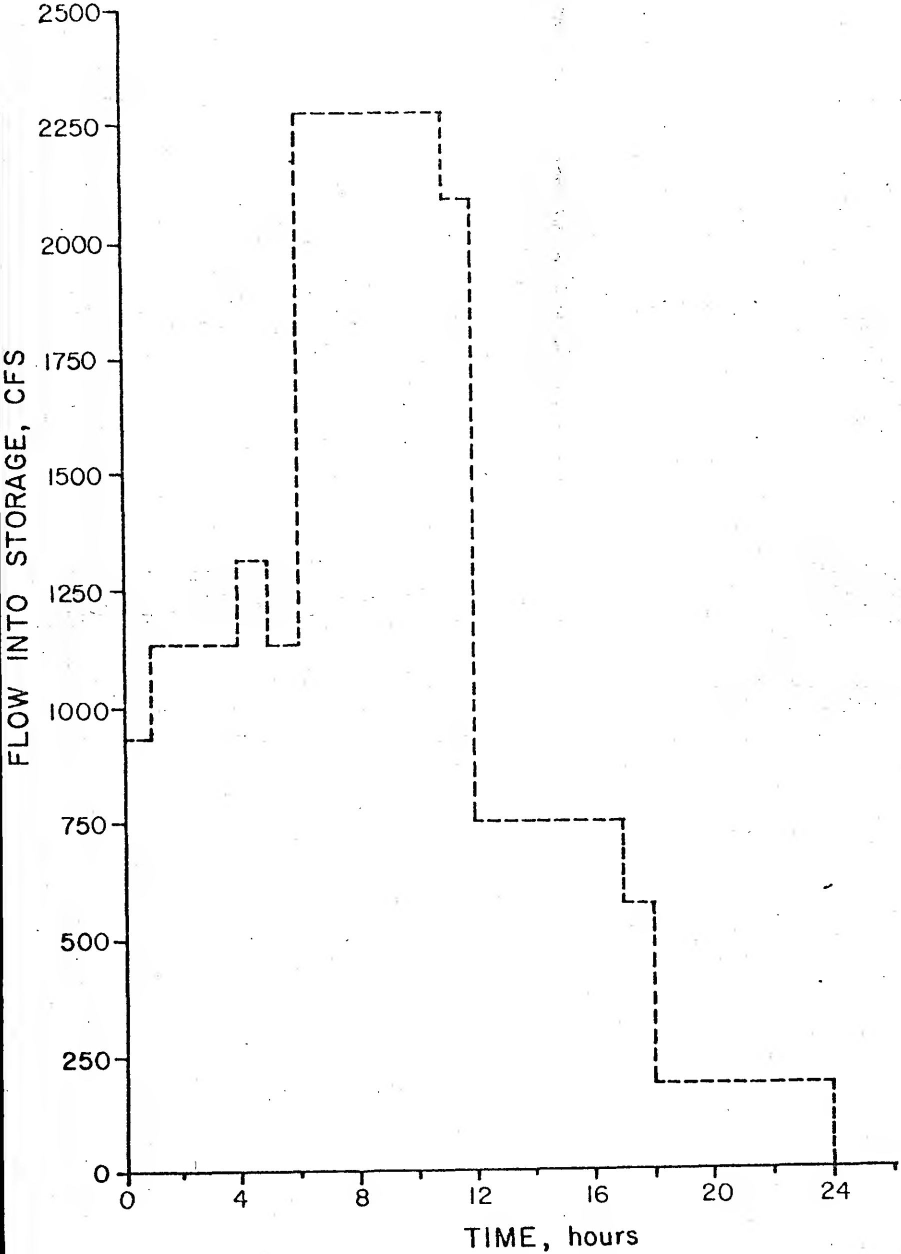


Figure 5-4. Input Flow Rate for Wet Weather Event No. 52.

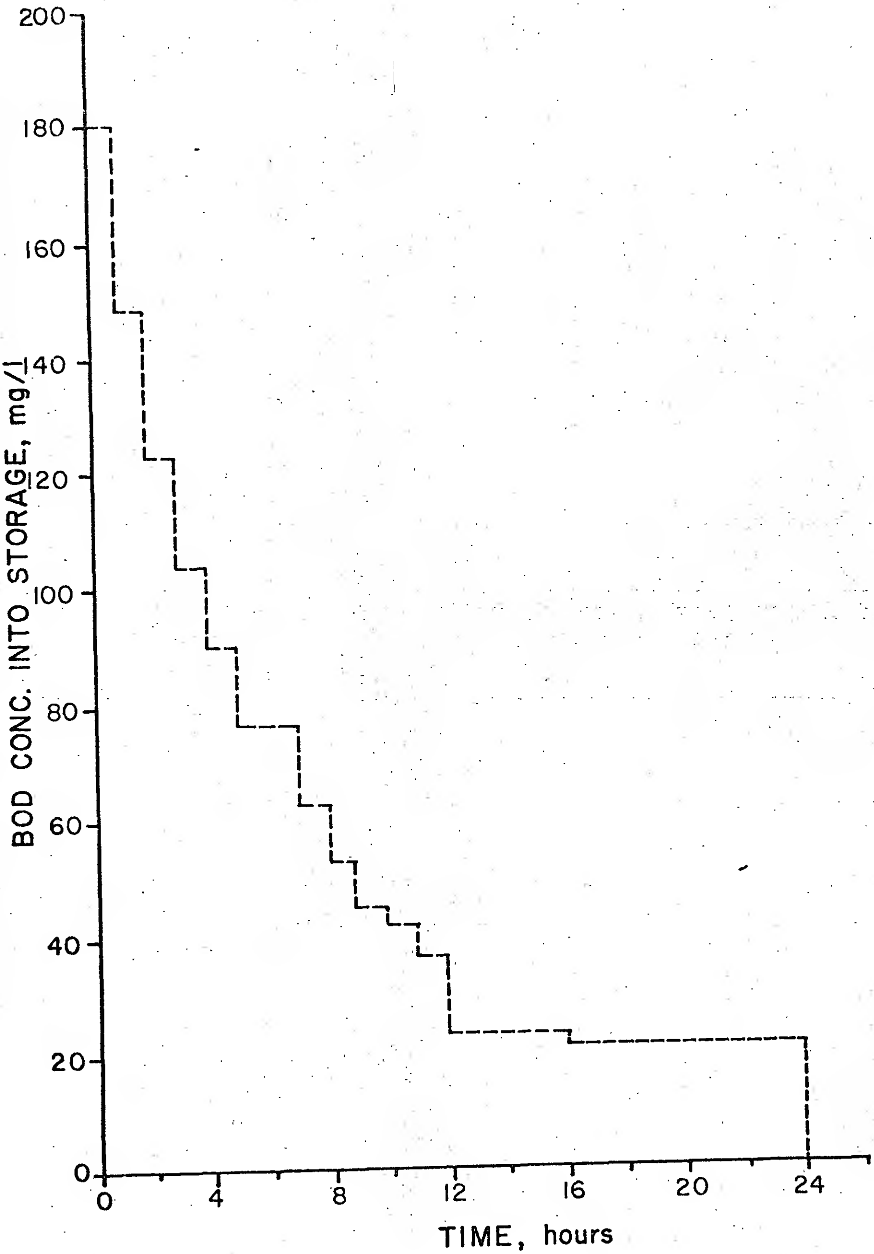


Figure 5-5. Input BOD Concentration for Wet Weather Event No. 52.

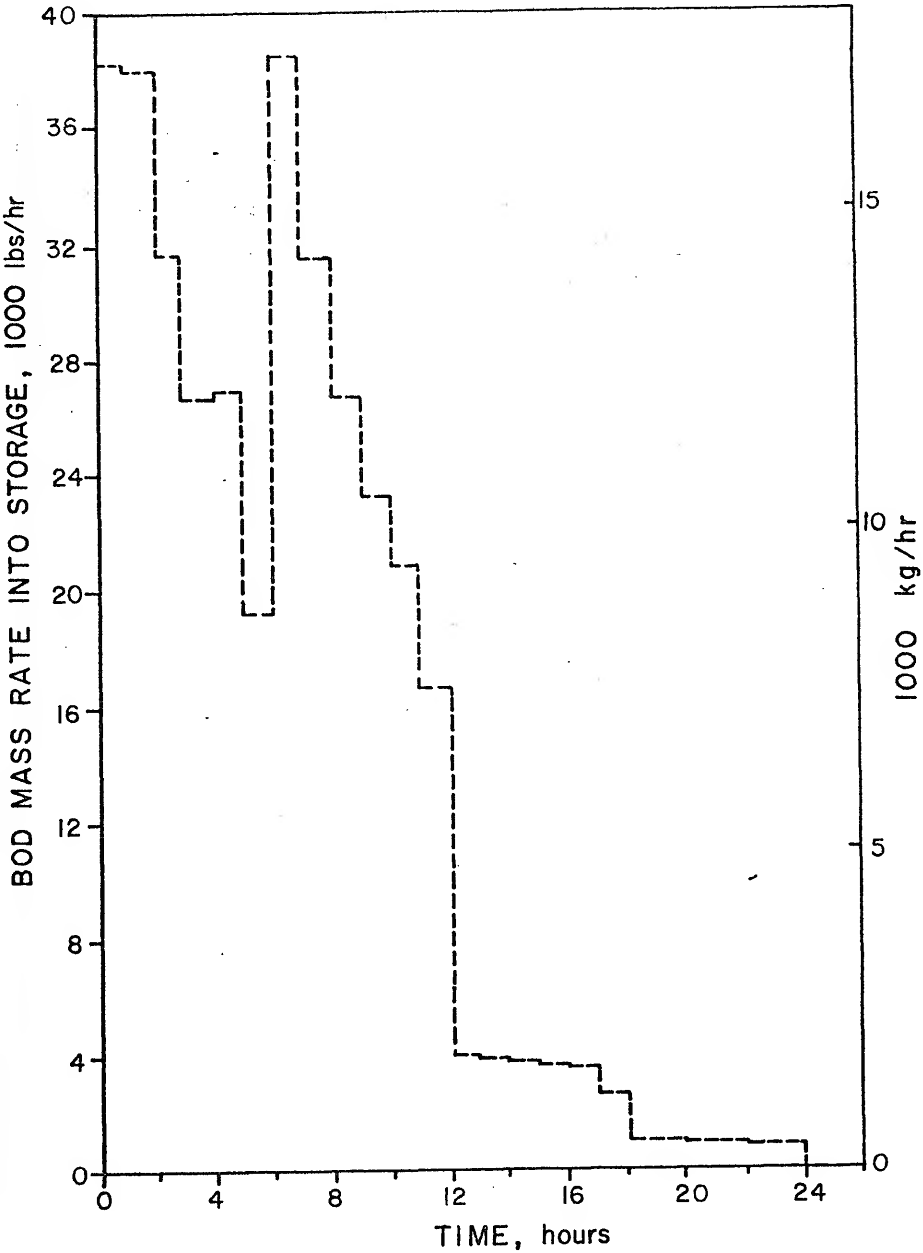


Figure 5-6. Input BOD Mass Rate for Wet Weather Event No. 52.

Table 5-3

Required Basin Volume* for Desired Average Detention Time**

Average Detention Time $\bar{t}_D$, Hours	Basin Volume V, Cubic Feet (cu m)
2	4,300,000 (121,776)
6	12,900,000 (366,328)
12	25,800,000 (730,656)
24	51,600,000 (1,461,312)

*One storage/treatment facility for entire urban area.

**As defined by equation (5.24) for all wet weather events.

generated by the study area is processed at a single central facility. In a well-mixed constant volume unit, the time-varying output flow rate is identical to the time-varying input flow rate, as shown by comparing Figure 5-4 to Figure 5-7. Figure 5-7 is the output runoff hydrograph regardless of the basin residence time. The response of the well-mixed constant volume model to BOD concentration and BOD mass rate inputs is shown, respectively, in Figures 5-8 and 5-9 for varied average detention time.

Figures 5-8 and 5-9 indicate that significant reductions in peak BOD concentrations and mass rates are obtained by increasing the residence time of inflows to the storage/treatment system. The residence time, of course, is increased by simply enlarging the capacity of the holding basin. It is useful to examine input and output statistics for the storm event, presented in Table 5-4. Percentage reductions in peak BOD concentrations and mass rates and mean BOD concentrations and mass rates are summarized in Table 5-5. From an examination of the standard deviations of the BOD mass rates, it appears that equalization of the effluent is being accomplished more efficiently as the average detention time increases. An alternative approach to computation of concentration variances, and a detailed analysis of equalization in the well-mixed constant volume model, are presented in

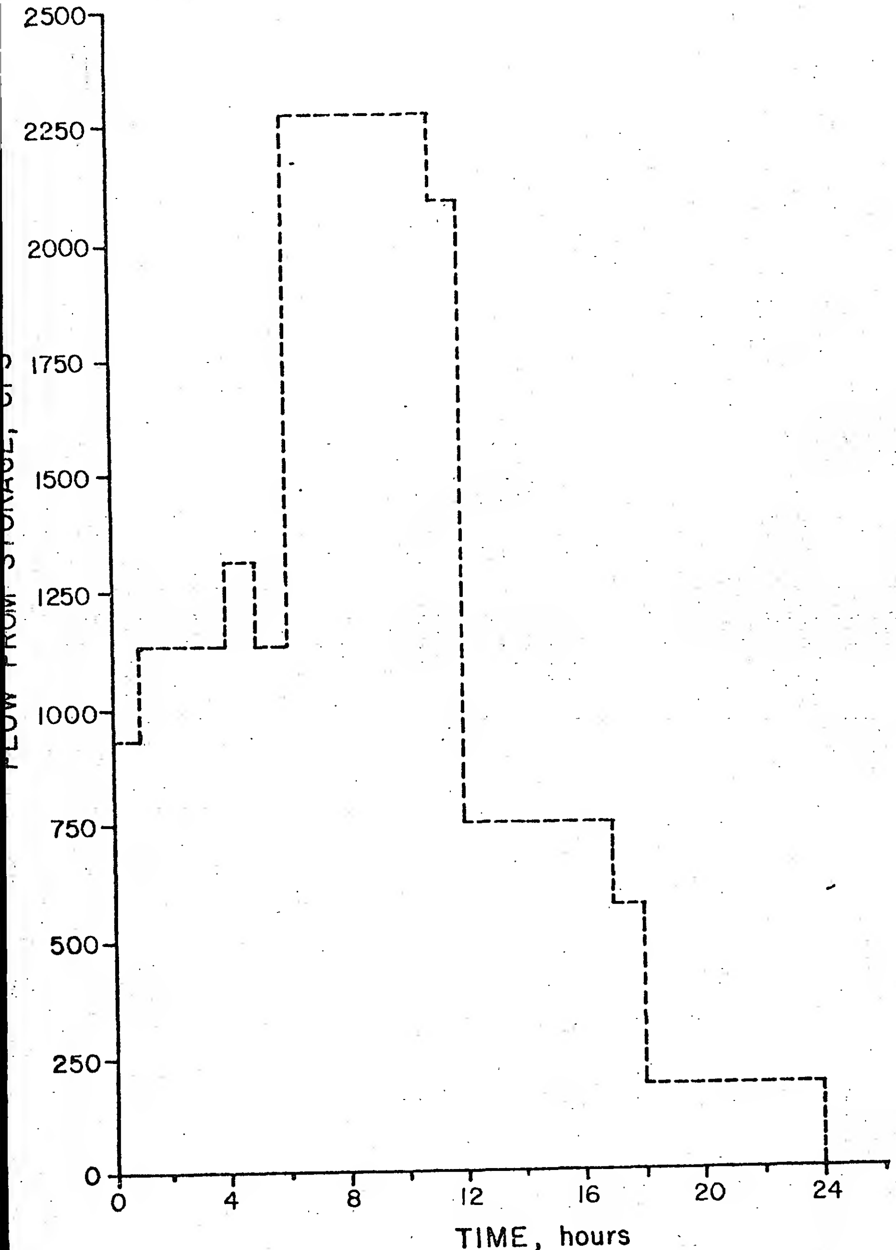


Figure 5-7. Output Flow Rate for Wet Weather Event No. 52.

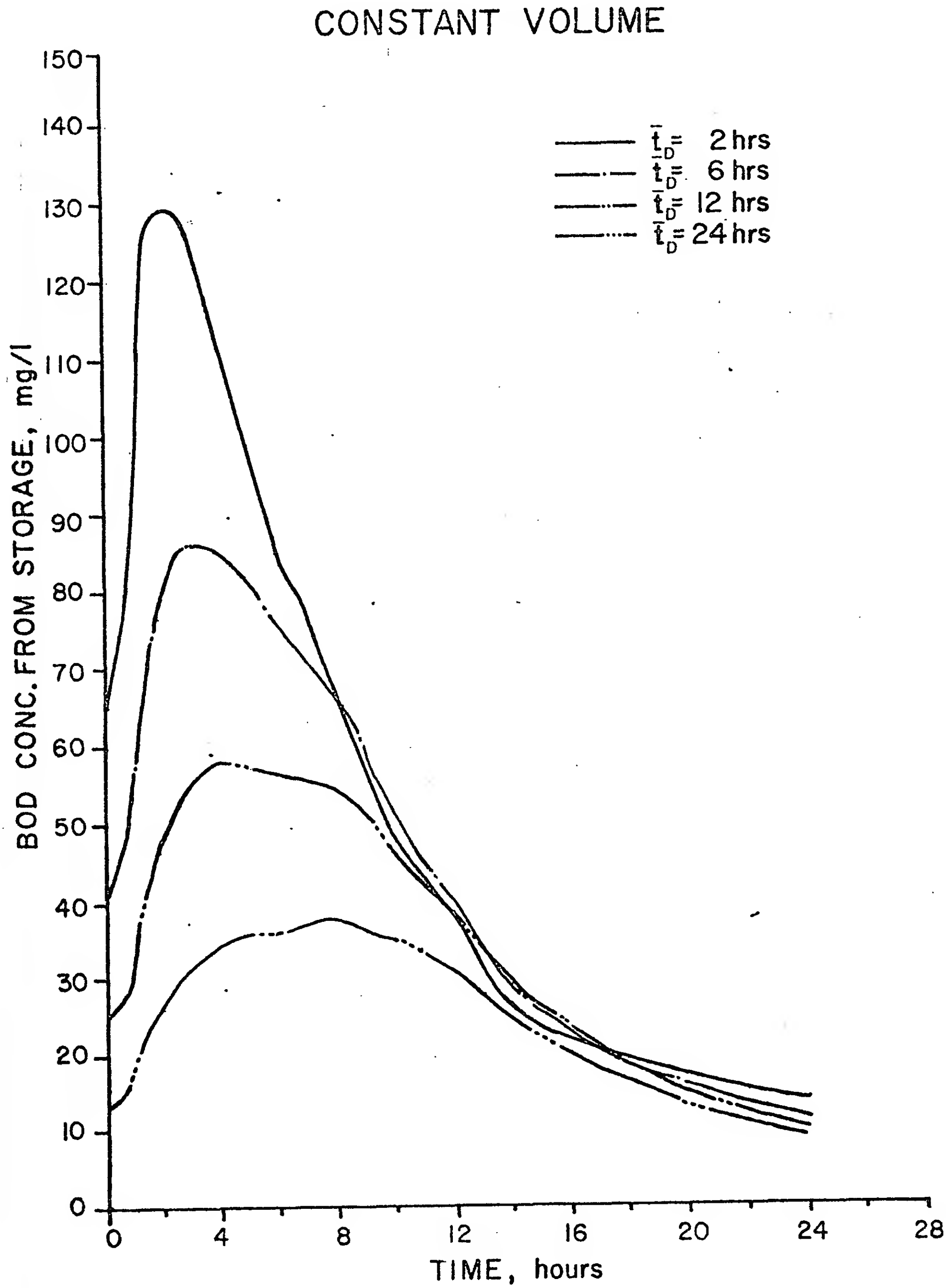


Figure 5-8. BOD Concentration Response for Wet Weather Event No. 52.

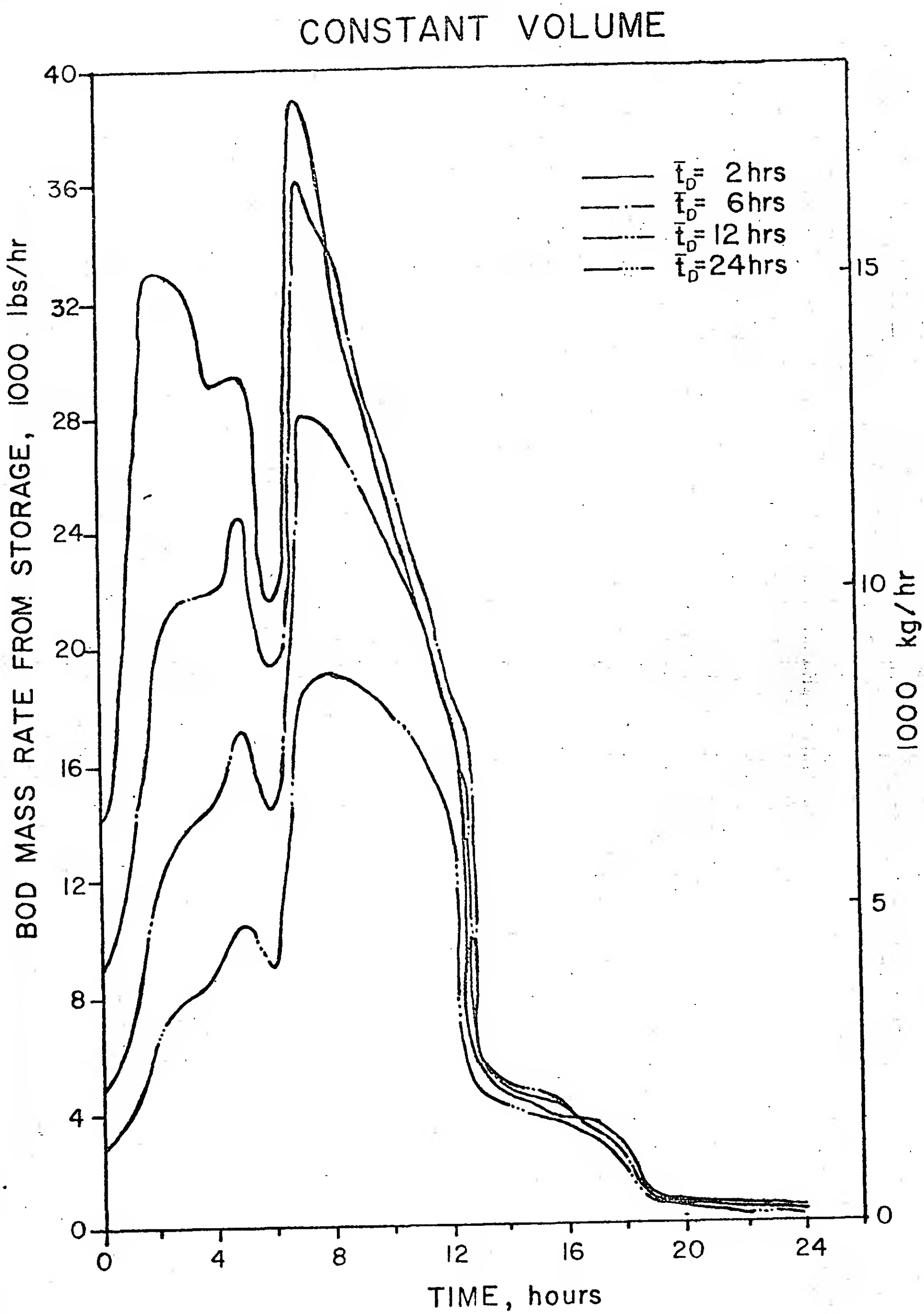


Figure 5-9. BOD Mass Rate Response for Wet Weather Event No. 52.

Table 5-4

Statistics for Well-Mixed Constant Volume Model for Wet Weather Event No. 52

Item	Input to Storage/Treatment	Output from Storage/Treatment System			
		$\overline{t}_D$ = 2 hrs	$\overline{t}_D$ = 6 hrs	$\overline{t}_D$ = 12 hrs	$\overline{t}_D$ = 24 hrs
Peak BOD Concentration	180 mg/l	130	86	58	37
Mean BOD Concentration*	63 mg/l	62	54	43	30
Peak Flow Rate	2267 cfs (64 cu m/sec)	2267 (64)	2267 (64)	2267 (64)	2267 (64)
Mean Flow Rate	1070 cfs (30 cu m/sec)	1070 (30)	1070 (30)	1070 (30)	1070 (30)
Standard Deviation of Flow Rate	771 cfs (22 cu m/sec)	771 (22)	771 (22)	771 (22)	771 (22)
Coefficient of Variation of Flow Rate	0.72	0.72	0.72	0.72	0.72
Peak BOD Mass Rate	38,499 lbs/hr (17,462 kg/hr)	38,857 (17,625)	36,203 (16,421)	28,096 (12,744)	19,016 (8,626)
Mean BOD Mass Rate	15,213 lbs/hr (6,900 kg/hr)	14,995 (6,802)	13,049 (5,919)	10,403 (4,719)	7,283 (3,304)
Standard Deviation of BOD Mass Rate	14,262 lbs/hr (6,469 kg/hr)	13,778 (6,250)	11,993 (5,440)	9,580 (4,345)	6,694 (3,036)
Coefficient of Variation of BOD Mass Rate	0.94	0.92	0.92	0.92	0.92

*Flow-weighted average

Table 5-5

Pollutant Removal Effectiveness of
Well-Mixed Constant Volume Model for Wet
Weather Event No. 52

Item	% Reduction for			
	$\bar{t}_D$=2 hrs	$\bar{t}_D$=6 hrs	$\bar{t}_D$=12 hrs	$\bar{t}_D$=24 hrs
Peak BOD Concentration	28	52	68	79
Mean BOD Concentration**	2	14	32	52
Peak BOD Mass Rate	-1*	6	27	51
Mean BOD Mass Rate	1	14	32	52

*Slight increase in peak BOD mass rate is observed because the time sequence of inflows and concentrations result in pollutant accumulation and the detention time is insufficient for decay.

**From flow-weighted average.

Chapter VI. From Table 5-5, it appears that an average detention time of at least 6 hours would be required to effect adequate pollutant removal for Wet Weather Event No. 52.

Displaying pollutographs for every wet weather event during the year would provide little additional information on the effectiveness of the storage/treatment reactor. Thus, it is convenient to resort to statistical techniques again to present results in a unified format. Frequency analyses are performed on BOD concentrations and mass rates for every hour of all wet weather events. Figure 5-10 represents the frequency distribution of all input BOD concentrations to the WWF storage/treatment facility. These concentrations include zero values for those periods of dry weather within the storm events (refer to Chapter IV). The frequency distribution of all BOD concentrations in the well-mixed basin, and in the discharge, is presented in Figure 5-11. A good basis for comparison is provided when cumulative frequencies are plotted together, as in Figure 5-12.

The effects of mixing and first-order decay in the system may be observed by comparing the cumulative frequency curve for input BOD concentrations to the curves representing the response for different residence times. As the detention time increases, a greater number of high concentrations mix with the low concentrations,

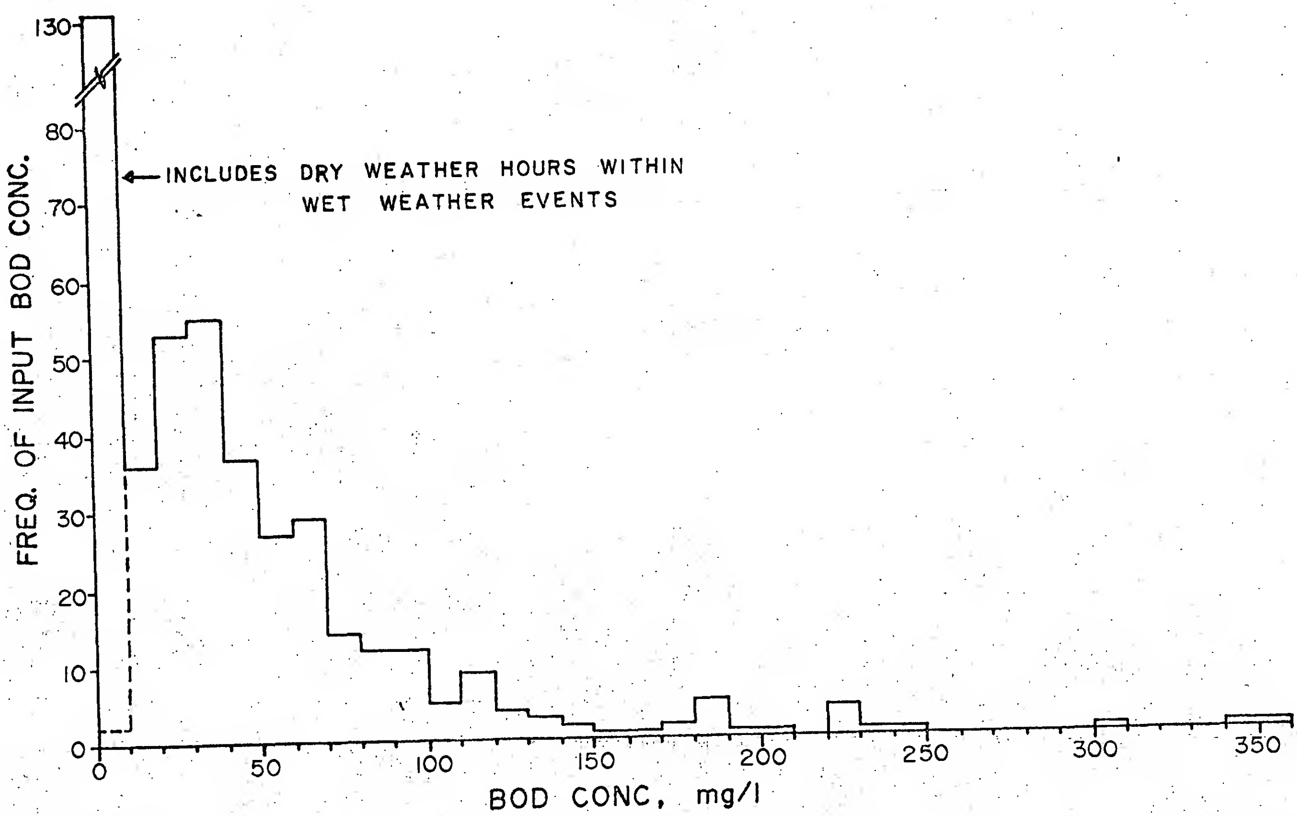


Figure 5-10. Frequency Distribution of Input BOD Concentrations for All Wet Weather Events.

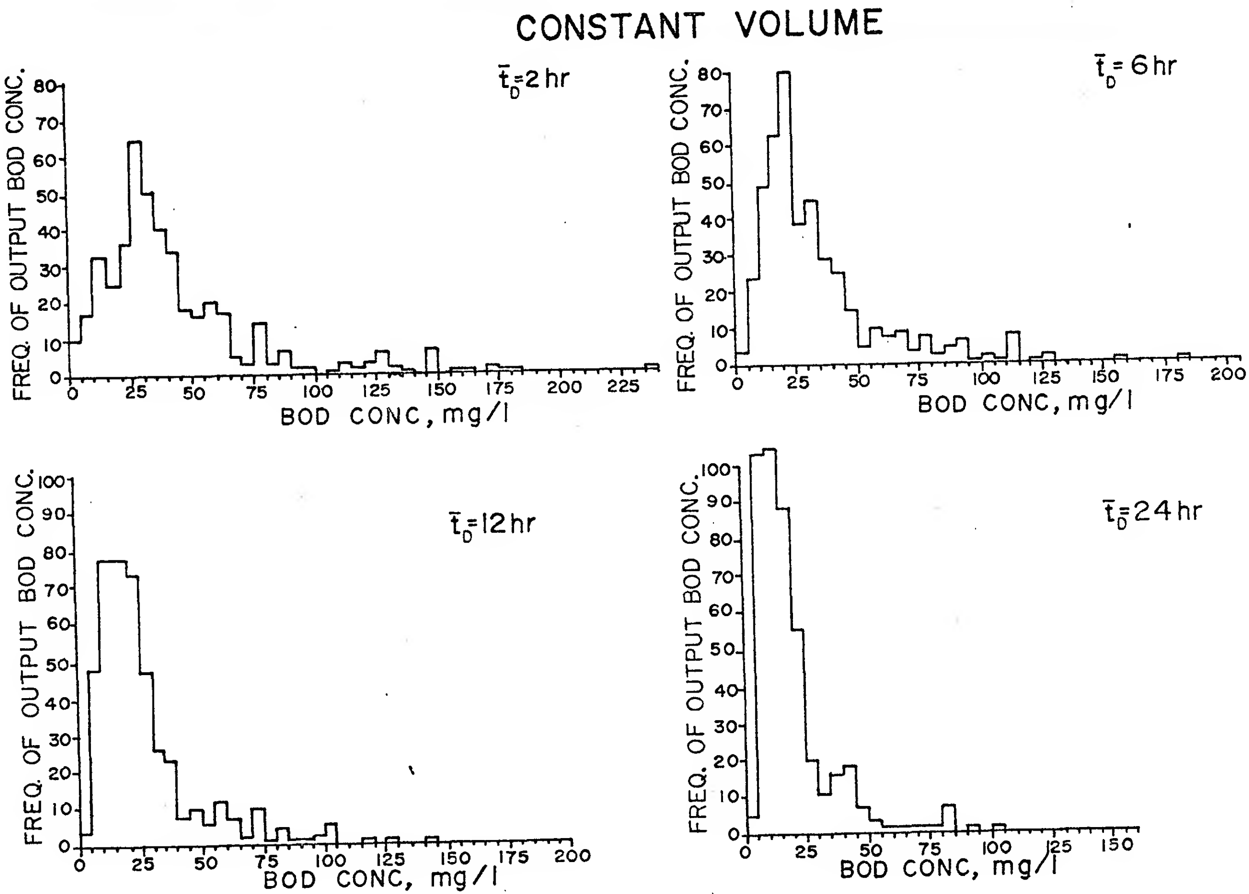


Figure 5-11. Frequency Distribution of Output BOD Concentration for All Wet Weather Events.

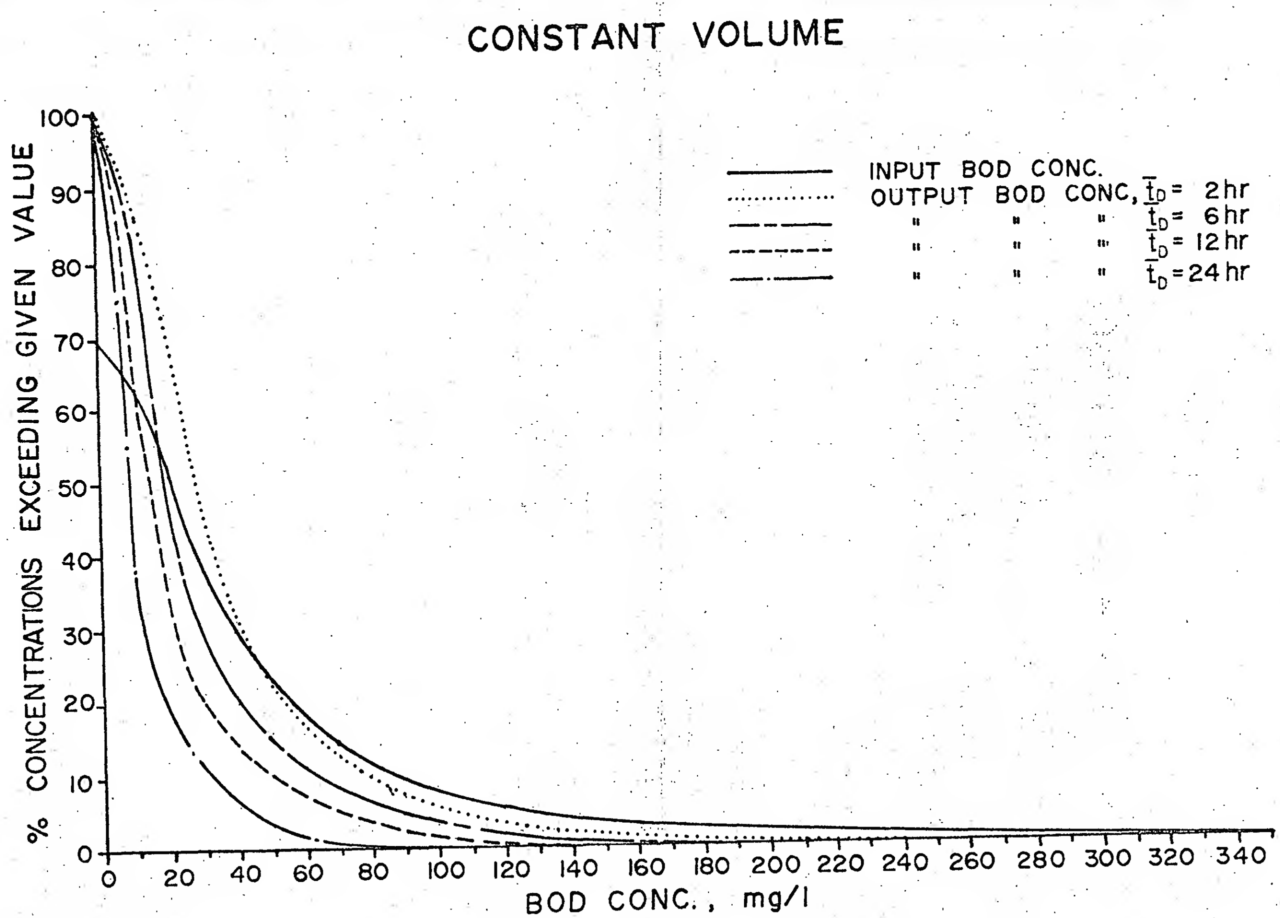


Figure 5-12. Cumulative Frequency of BOD Concentrations for All Wet Weather Events.

resulting in a redistribution of magnitudes. Similar analyses are performed for the BOD mass rates. The frequency distribution of input BOD mass rates for all wet weather events is shown in Figure 5-13. Likewise, the frequency distribution of the output BOD mass rates is presented in Figure 5-14 for various detention times. Large variations in maximum BOD mass rates and the higher BOD mass rates precluded the use of a common scale for all graphs. The cumulative frequencies are plotted together in Figure 5-15. A relatively uniform reduction in the magnitude of all BOD mass rates in the output stream is observed as the holding time increases. To supplement these figures, input and output statistics for all wet weather events are provided in Table 5-6. The pollutant removal effectiveness of the well-mixed constant volume model is summarized in Table 5-7. It seems, again, that adequate attenuation of extreme mass flow rates requires at least 6 hours of detention time. Reductions in mean BOD concentration and mean mass rate are also quite adequate for $\bar{t}_D \geq 6$ hours.

5.3 Well-Mixed Variable Volume Model

Continuous Forcing Functions

The storage/treatment facility for wet weather flows may be modeled as a well-mixed variable volume basin,

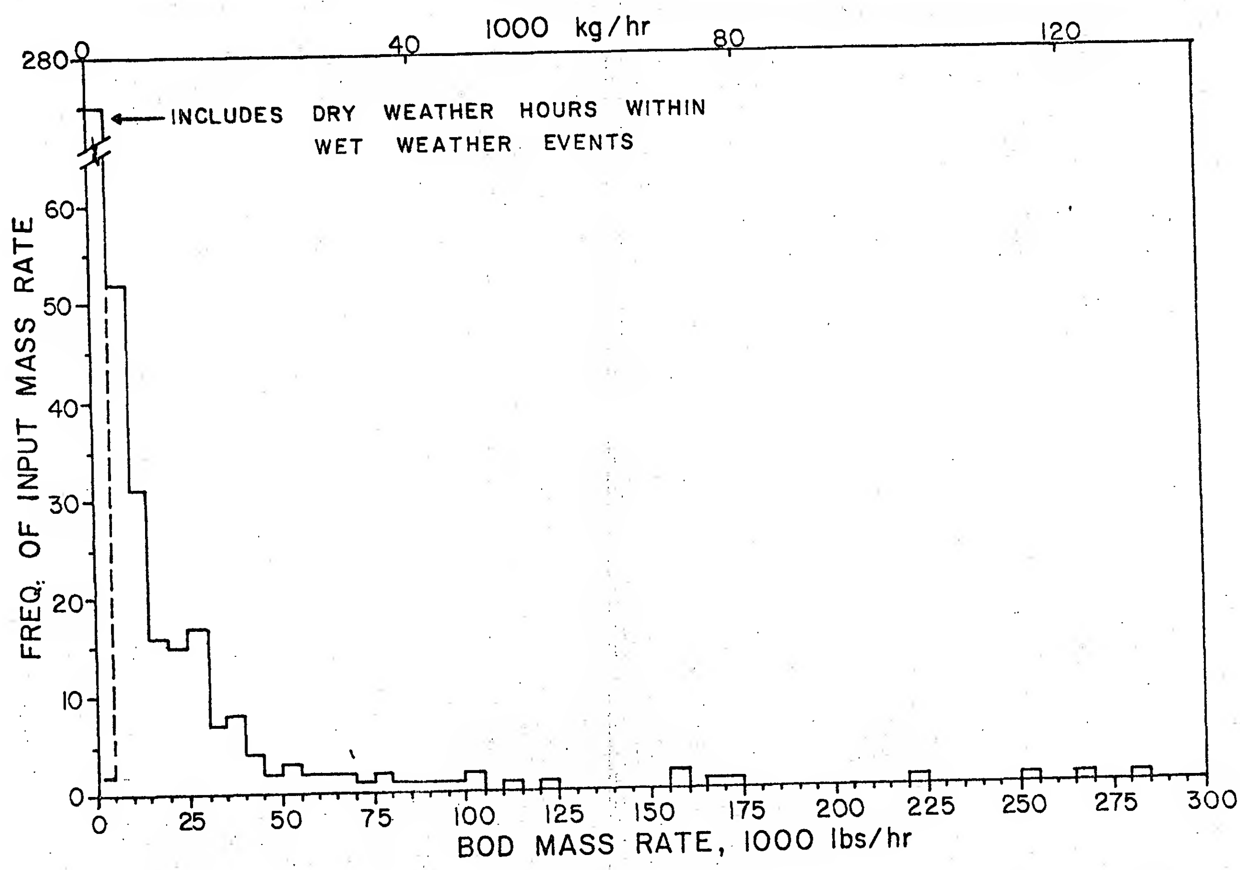


Figure 5-13. Frequency Distribution of Input BOD Mass Rate for All Wet Weather Events.

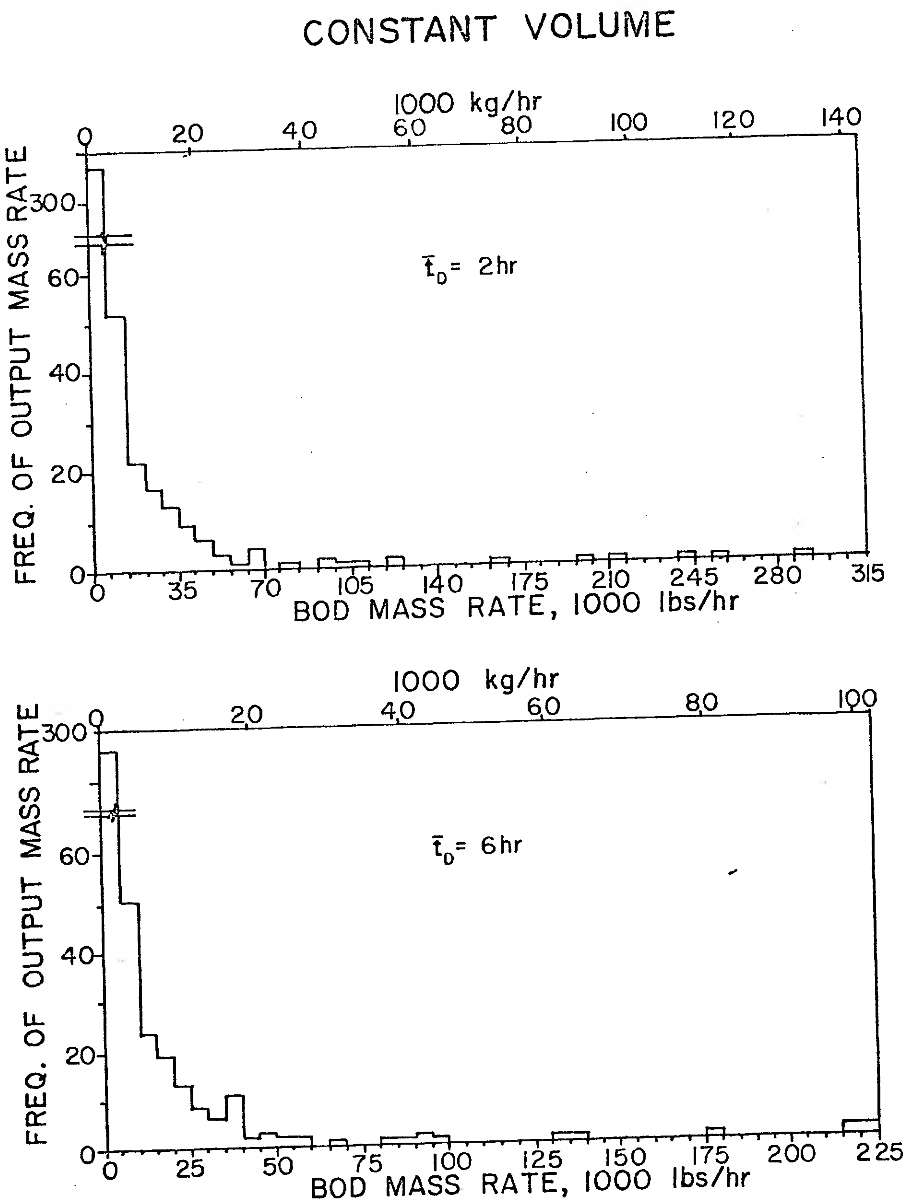


Figure 5-14. Frequency Distribution for Output BOD Mass Rate for All Wet Weather Events.

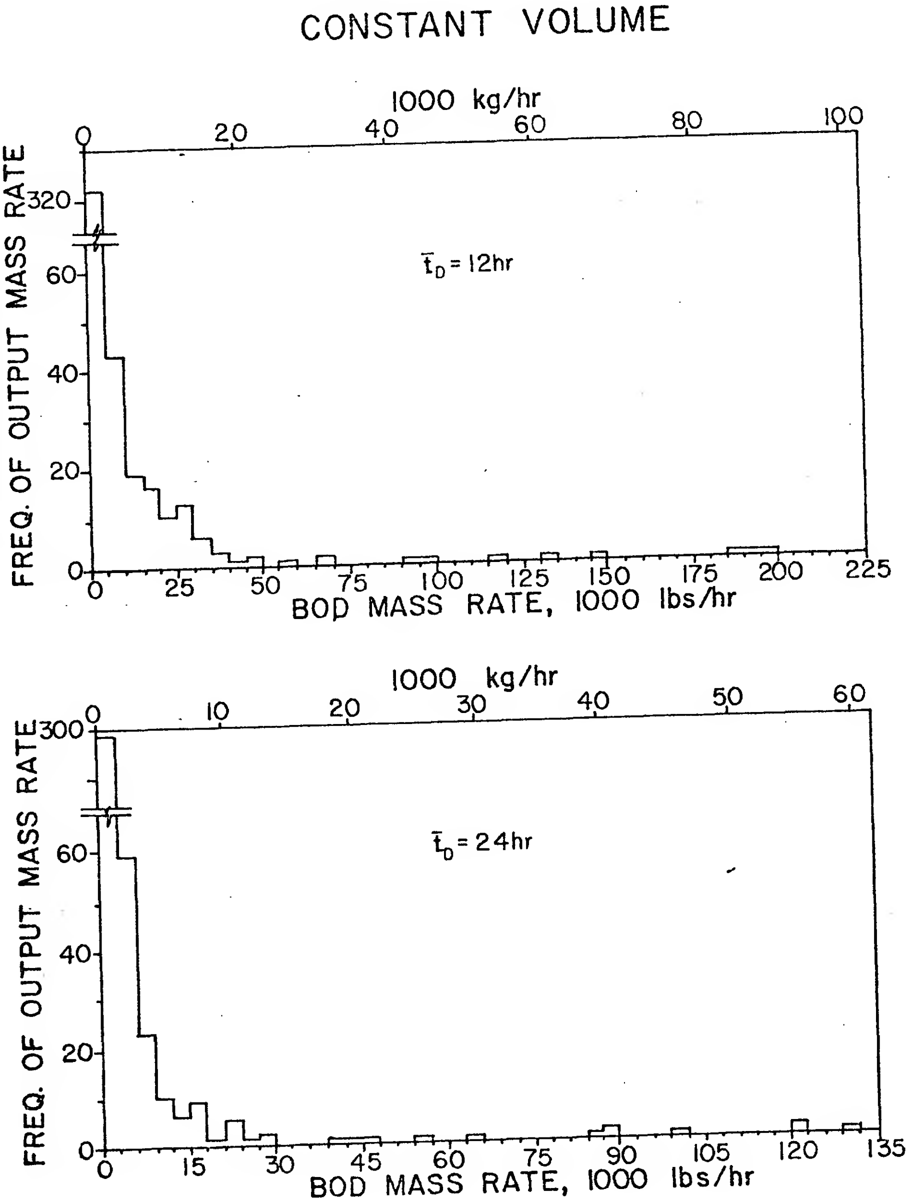


Figure 5-14 continued

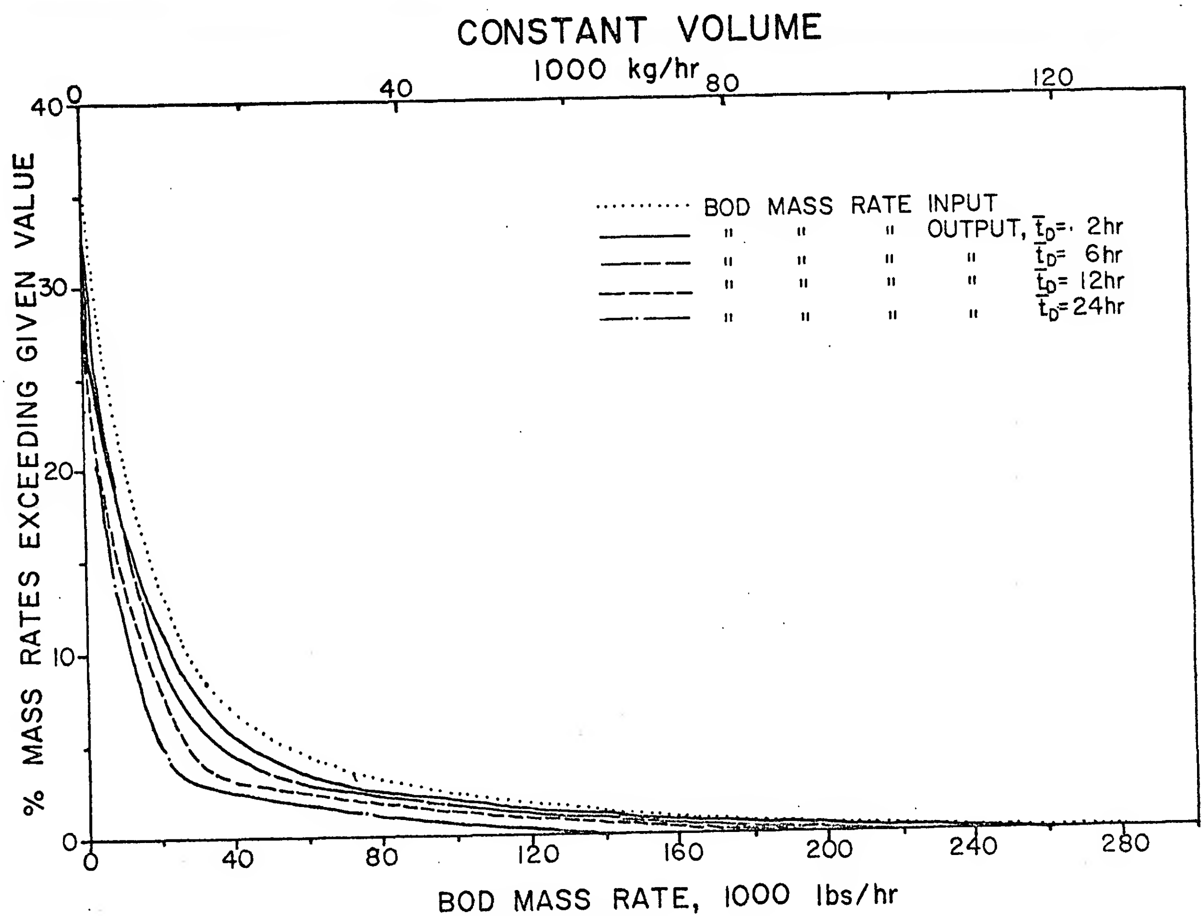


Figure 5-15. Cumulative Frequency of BOD Mass Rates for All Wet Weather Events.

Table 5-6

Statistics for Well-Mixed Constant Volume Model for All Wet Weather Events

	Input to Storage/Treatment	Output from Storage/Treatment System			
		$\bar{t}_D$ = 2 hrs	$\bar{t}_D$ = 6 hrs	$\bar{t}_D$ = 12 hrs	$\bar{t}_D$ = 24 hrs
Peak BOD Concentration	352 mg/l	242	187	143	100
Mean BOD Concentration*	64 mg/l	49	37	28	19
Peak Flow Rate	24,940 cfs (706 cu m/sec)	24,940 (706)	24,940 (706)	24,940 (706)	24,940 (706)
Mean Flow Rate	1,584 cfs (45 cu m/sec)	1,584 (45)	1,584 (45)	1,584 (45)	1,584 (45)
Standard Deviation of Flow Rate	1,290 cfs (37 cu m/sec)	1,290 (37)	1,290 (37)	1,290 (37)	1,290 (37)
Coefficient of Variation of Flow Rate	0.81	0.81	0.81	0.81	0.81
Peak BOD Mass Rate	285,563 lbs/hr (129,529 kg/hr)	298,141 (135,234)	232,174 (105,312)	200,396 (90,898)	146,950 (66,655)
Mean BOD Mass Rate	21,717 lbs/hr (9,851 kg/hr)	19,032 (8,633)	15,196 (6,893)	11,840 (5,371)	8,240 (3,738)
Standard Deviation of BOD Mass Rate	18,250 lbs/hr (8,278 kg/hr)	16,181 (7,340)	12,888 (5,846)	10,027 (4,548)	7,039 (3,193)
Coefficient of Variation of BOD Mass Rate	0.84	0.85	0.85	0.85	0.85

*Flow-weighted average.

Table 5-7

Pollutant Removal Effectiveness of
Well-Mixed Constant Volume Model for
All Wet Weather Events

	% Reduction for Given Detention			
Item	$\bar{t}_D$=2 hrs	$\bar{t}_D$=6 hrs	$\bar{t}_D$=12 hrs	$\bar{t}_D$=24 hrs
Peak BOD Concentration	31	47	59	72
Mean BOD Concentration *	23	42	56	70
Peak BOD Mass Rate	-4**	19	30	49
Mean BOD Mass Rate	12	30	45	62

*Flow-weighted average.

**Slight increase in peak BOD mass rate is due to pollutant accumulation while insufficient detention time is allowed for decay, because of the time sequence in flows and concentrations.

in which case the system may be represented as shown in Figure 5-16. The volumetric rates of flow into and out of the system are unequal functions of time. The pollutant mass in the tank is assumed to undergo first-order decay. The concentration forcing function, $c_1(t)$, and the response, $c_2(t)$, are both continuous functions of time. From a flow balance across the basin, the time rate of change in volume is expressed in differential form (Henderson, 1966).

$$\frac{dV}{dt} = Q_1(t) - Q_2(t) \tag{5.26}$$

where $Q_1(t)$ = fluid flow rate into tank, as a continuous function of time, L^3/T, and

$Q_2(t)$ = fluid flow rate out of the tank, as a continuous function of time, L^3/T.

It is further assumed that the outflow is a linear function of the volume in the basin to allow a solution to the continuity equation, thus

$$Q_2(t) = \alpha V(t) \tag{5.27}$$

where α = time constant, $1/T$, and

$V(t)$ = time-varying volume of the basin, L^3.

From equations (5.26) and (5.27), an expression for the outflow as a function of the inflow, in differential form,

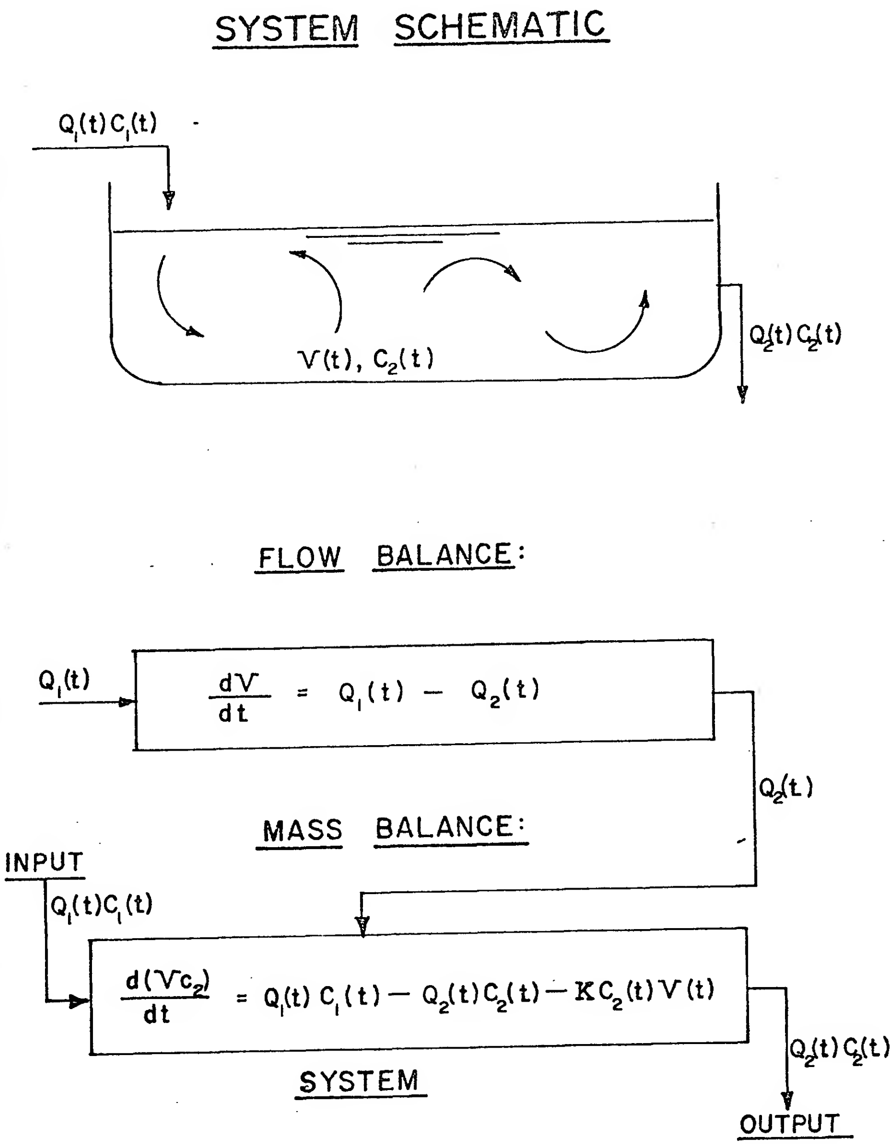


Figure 5-16. Well-Mixed Variable Volume Model.

may be obtained.

$$\frac{dQ_2}{dt} + \alpha Q_2(t) = \alpha Q_1(t) \qquad (5.28)$$

where $Q_1(t)$ is a known function of time. Therefore, equation (5.28) is directly integrable by applying an integrating factor (Ross, 1964).

$$Q_2(t) = \alpha e^{-\alpha t} \int_0^t Q_1(t) e^{\alpha t}\, dt + Q_2(t=0) e^{-\alpha t} \qquad (5.29)$$

where $Q_2(t=0)$ = initial outflow of the system, L^3/T.

From a mass balance of the dissolved material across the holding facility, the governing equation is found to be

$$\frac{d(Vc_2)}{dt} = Q_1(t)c_1(t) - Q_2(t)c_2(t) - Kc_2(t)V(t) \qquad (5.30)$$

From equation (5.27), the volume in equation (5.30) may be expressed as a function of the outflow rate. Thus,

$$\frac{1}{\alpha} Q_2(t) \frac{dc_2}{dt} + c_2(t) \left[\frac{1}{\alpha}\ \frac{dQ_2}{dt}\right] =$$

$$Q_1(t)c_1(t) - \left[\frac{\alpha + K}{\alpha}\right] Q_2(t)c_2(t) \qquad (5.31)$$

Rearranging equation (5.31) into a more convenient form, the following relationship is obtained:

$$\frac{dc_2}{dt} + \left[\frac{1}{Q_2(t)}\left(\frac{dQ_2}{dt}\right) + \alpha + K\right] c_2(t) = \frac{\alpha Q_1(t) c_1(t)}{Q_2(t)} \qquad (5.32)$$

Equation (5.32) is a linear, first-order, ordinary differential equation with variable coefficients. Letting

$$P(t) = \frac{1}{Q_2(t)}\left(\frac{dQ_2}{dt}\right) + \alpha + K \qquad (5.33)$$

the solution to equation (5.32) is

$$c_2(t) = e^{-\int_0^t P(t)dt} \left[\int_0^t e^{\int_0^t P(t)dt} \frac{\alpha Q_1(t) c_1(t)}{Q_2(t)} dt + c_2(t{=}0)\right] \qquad (5.34)$$

Integration of equation (5.34) is dependent on the form of, essentially, three forcing functions: $c_1(t)$, $Q_1(t)$, and $Q_2(t)$ which may be obtained from $Q_1(t)$. For non-trivial forcing functions, integration of equation (5.34) by direct analysis is not possible. Two examples are presented in Table 5-8. For either case, a numerical

Table 5-8

Integration of Equation (5.34) for Various Forcing Functions of Concentration and Flow Rates*

Form of Forcing Functions	Integration of $c_2(t)$
Exponential $c_1(t) = c_1 e^{+bt}$ Triangular $Q_1(t) = 4Qt(1-t)$ Combination $Q_2(t) = 4Q(1+\frac{2}{\alpha})(\frac{1}{\alpha})\cdot [\alpha t-1+e^{-\alpha t}] - 4Qt^2$	$\exp\left[-\int_0^t \left(\frac{\alpha^2+2\alpha-\alpha^2 e^{-\alpha t}-2\alpha e^{-\alpha t}-2\alpha^2 t}{\alpha e^{-\alpha t}+2e^{-\alpha t}-\alpha^2 t^2+\alpha^2 t+2\alpha t-\alpha-2} +\alpha+ K\right)dt\right]\cdot$ $\left[\int_0^t \exp\left[\int_0^t\left(\frac{\alpha^2+2\alpha-\alpha^2 e^{-\alpha t}-2\alpha e^{-\alpha t}-2\alpha^2 t}{\alpha e^{-\alpha t}+2e^{-\alpha t}-\alpha^2 t^2+\alpha^2 t+2\alpha t-\alpha-2} +\alpha+ K\right)dt\right]\cdot \left[\frac{\alpha^3 c_1(t - t^2)\, e^{+bt}}{(\alpha+2)e^{-\alpha t}-\alpha^2 t^2+\alpha^2 t+2\alpha t-\alpha-2}\right] dt+c_2(t=0)\right]$
Exponential $c_1(t) = c_1 e^{-bt}$ Exponential $Q_1(t) = Q e^{-at}$ Exponential $Q_w(t) = \frac{\alpha Q}{\alpha-a}[e^{-at} - e^{-\alpha t}] + Qe^{-\alpha t}$	$\exp -\left[\int_0^t \left(\frac{\alpha^2 e^{-at}+\alpha K e^{-at}-a\alpha e^{-at}-aKe^{-\alpha t}}{\alpha e^{-at}-ae^{-\alpha t}}\right) dt\right]\cdot$ $\left[\int_0^t \exp\left[\int_0^t \left(\frac{\alpha^2 e^{-at}+\alpha K e^{-at}-a\alpha e^{-at}-aKe^{-\alpha t}}{\alpha e^{-at}-ae^{-\alpha t}}\right) dt\right]\cdot \left[\frac{\alpha\,(\alpha-a)c_1 e^{-(a+b)t}}{\alpha e^{-at}-ae^{-\alpha t}}\right] dt + c_2\,(t=0)\right]$

*Unequal inflow and outflow rates, variable volume detention facility, where

$$Q_2(t) = \alpha\, V(t).$$

approximation of the original differential equation, equation (5.32), or a numerical integration of equation (5.34) is necessary.

Discrete Inputs

The response of the well-mixed variable volume model to discrete inputs is derived from the basic equations developed for continuous forcing functions. Forcing functions are assumed to be step function inputs, as before. Thus, from equation (5.29) and after integration over the time step length, Δt,

$$Q_2(t_i + \Delta t) = Q_i\ [1 - \exp(-\alpha\ \Delta t)] + Q_2(t_i)\exp(-\alpha\ \Delta t) \tag{5.35}$$

where $Q_2(t_i + \Delta t)$ = output flow rate at the end of the time step, L^3/T,

Q_i = value of input flow rate from t_i to $(t_i + \Delta t)$, L^3/T, and

$Q_2(t_i)$ = outflow value at the end of the previous time step, L^3/t.

The average outflow rate for each time step may be obtained by integrating equation (5.35) over the length of the time step. That is,

$$\overline{Q}_2(t_i + \frac{\Delta t}{2}) = \frac{1}{\Delta t}\int_0^{\Delta t} Q_2(\tau)d\tau \tag{5.36}$$

and after appropriate substitution and subsequent integration,

$$\overline{Q}_2(t_i + \frac{\Delta t}{2}) = Q_i + \frac{Q_i - Q_2(t_i)}{\alpha} [\exp(-\alpha \, \Delta t) - 1] \qquad (5.37)$$

where $\overline{Q}_2(t_i + \frac{\Delta t}{2})$ = time-averaged outflow rate for each time step, L^3/T.

To obtain the discrete form of equation (5.34) some preliminary analysis is required. For convenience, let Δt be replaced by τ, Q_i by a, $Q_2(t_i)$ by b, and $Q_2(t_i + \Delta t)$ by $Q_2(\tau)$ in equation (5.35). Thus, rearranging terms,

$$Q_2(\tau) = a + b \exp(-\alpha\tau) \qquad (5.38)$$

Then,

$$\frac{dQ_2}{d\tau} = - \ b \exp(-\alpha\tau) \qquad (5.39)$$

and by substituting equations (5.38) and (5.39) into (5.33),

$$P(\tau) = \frac{-\alpha \ b \ e^{-\alpha\tau}}{a + b \ e^{-\alpha\tau}} + \alpha + K \qquad (5.40)$$

It is now possible to begin the derivation of equation (5.34) by solving the nonconstant terms,

$$\int_0^t P(\tau)\, d\tau = \ell n \left[a + b\, e^{-\alpha t}\right]$$

$$- \ \ell n\,(a + b) + \alpha t + Kt \qquad (5.41)$$

Thus,

$$\exp\left[-\int_0^t P(\tau)\, d\tau\right] = \frac{a + b}{a + b\, e^{-\alpha t}} \exp\,(-\alpha t - Kt) \qquad (5.42)$$

and

$$\exp\left[\int_0^t P(\tau)\, d\tau\right] = \frac{a + b\, e^{-\alpha t}}{a + b} \exp\,(\alpha t + Kt) \qquad (5.43)$$

therefore,

$$\int_0^t \exp\left[\int_0^t P(\tau) d\tau\right] \frac{1}{Q_2(\tau)}\, d\tau =$$

$$\frac{1}{(a + b)(\alpha + K)} \left[e^{(\alpha + K)t} - 1\right] \qquad (5.44)$$

Substituting equations (5.42) through (5.44) into equation (5.34), replacing a and b by their original variables, and assuming step function inputs, the discrete form of equation (5.34) is given by

$$c_2(t_i + \Delta t) = \frac{\alpha}{\alpha + K} \left[\frac{Q_i c_i}{Q_i - [Q_i - Q_2(t_i)]\exp(-\alpha\, \Delta t)} \right] \cdot$$

$$[1 - \exp(-\alpha\, \Delta t - K\, \Delta t)] \ +$$

$$\frac{Q_2(t_i) c_2(t_i)}{Q_i - [(Q_i - Q_2(t_i)]\exp(-\alpha\, \Delta t)} \exp(-\alpha\, \Delta t - K\, \Delta t)$$

$$(5.45)$$

The average concentration for each time step is theoretically obtained by integrating equation (5.45) over the length of the time step.

$$\bar{c}_2(t_i + \frac{\Delta t}{2}) = \frac{1}{\Delta t}\int_0^{\Delta t} c_2(\tau)d\tau \qquad (5.46)$$

However, an exact integration of equation (5.46) is impossible. Therefore, the average mass rate output is obtained from

$$\bar{W}_2(t_i + \frac{\Delta t}{2}) = \frac{1}{\Delta t}\int_0^{t} c_2(\tau)Q_2(\tau)d\tau \qquad (5.47)$$

where $c_2(\tau)$ is given by equation (5.45) and $Q_2(\tau)$ is given by equation (5.35). This expression may be integrated because some complicated terms are canceled out. The integration of equation (5.47) yields

$$\bar{W}_2(t_i + \frac{\Delta t}{2}) = \frac{1}{(\alpha + K)\Delta t}\Big[\alpha Q_i c_i \Delta t + \frac{\alpha}{\alpha + K} \cdot$$
$$[\exp(-\alpha\Delta t - K\Delta t) - 1]Q_i c_i - Q_2(t_i)c_2(t_i) \cdot$$
$$[\exp(-\alpha\Delta t - K\Delta t) - 1]\Big] \qquad (5.48)$$

where $\bar{W}_2(t_i + \frac{\Delta t}{2})$ = time-averaged output mass rate for each time step, M/T.

The discrete mass rate input is given by equation (5.17),

as for the constant volume storage/treatment model. The average concentration for each time step is then obtained from equation (5.48) as follows:

$$\overline{c}_2(t_i + \frac{\Delta t}{2}) = \frac{\overline{W}_2(t_i + \frac{\Delta t}{2})}{\overline{Q}_2(t_i + \frac{\Delta t}{2})} \tag{5.49}$$

Unlike the constant volume model, the well-mixed variable volume facility has a constant detention time.

$$t_D = \frac{V(t_i)}{Q_2(t_i)} \tag{5.50}$$

since by equation (5.27)

$$t_D = \frac{V(t_i)}{\alpha\ V(t_i)} = \frac{1}{\alpha} \tag{5.51}$$

Thus, the volume in the facility varies per time step, but the outflow is a linear function of the basin volume such that every parcel of water entering the storage/treatment system will remain in the system for a time length of $1/\alpha$.

Interevent Processes

First-order decay between storm events is described by equation (5.25). However, when the storage/treatment system is represented by a variable volume model,

it is necessary to consider the outflow rate from the system several hours beyond the time of the last inflow. Thus, initial conditions of system outflow are defined by

$$Q_0(I) = Q_2(I-1)\exp[-\alpha \, DWT(I)] \tag{5.52}$$

where $Q_0(I)$ = system outflow rate at t=0, for storm event I, L^3/T, and

$Q_2(I-1)$ = outflow rate from the storage/treatment system during the last hour of runoff of the previous storm event, L^3/T.

Model Application

As stated previously, the detention time of inflows to the variable volume storage/treatment system is constant. The average storage volumes used by the holding facility, for fixed detention times, were obtained from digital computer simulation of all wet weather events and are presented in Table 5-9. Comparing Tables 5-3 and 5-9, it appears that the variable volume model is considerably more efficient in storage utilization as far as providing detention time to inflows is concerned.

Discrete inputs of flow, BOD concentration and BOD mass rate for Wet Weather Event No. 52 have been shown in Figures 5-4, 5-5 and 5-6. The response of the well-mixed variable volume model to these forcing functions

Table 5-9

Average Storage Volumes* Used
for Given Dentention Times**

Fixed Detention Time t_D, Hours	Average Basin Volume $\overline{V}$, Cubic Feet (Cubic Meters)
2	2,287,368 (64,778)
6	2,746,323 (77,776)
12	5,588,860 (158,277)
24	11,154,820 (315,905)

*One variable volume storage/treatment facility for the entire urban area's wet weather flows.

**Defined by equation (5.51).

is shown, respectively, in Figures 5-17, 5-18, and 5-19 for varied detention time. Figure 5-17 indicates, as expected, that significant reductions in peak flow rates out of the holding facility are obtained by increasing residence times of inflows to the system. The variable volume basin, however, does not provide significant reduction in BOD concentrations, as noted in Figure 5-18. On the other hand, because peak flow rates are reduced, BOD mass rate peaks are correspondingly decreased as observed in Figure 5-19. It is again useful to examine input and output statistics computed for the storm event, presented in Table 5-10. Pollutant removal efficiencies for peak BOD concentrations and mass rates and mean BOD concentrations and mass rates are summarized in Table 5-11.

Although Figure 5-18 indicates that the system response is practically identical for varied detention time, a flow-weighted average of output BOD concentrations reflects more accurately the impact to the receiving body of water. That is, the flows in the high concentration range are decreased while the flows in the low concentration range are increased, with the effect becoming more pronounced as the detention time increases. This phenomenon can be visualized by simply comparing the hydrographs in Figure 5-17 to the pollutographs of Figure 5-18. Thus, it is not surprising that when the flow-weighted average input BOD concentration is compared to

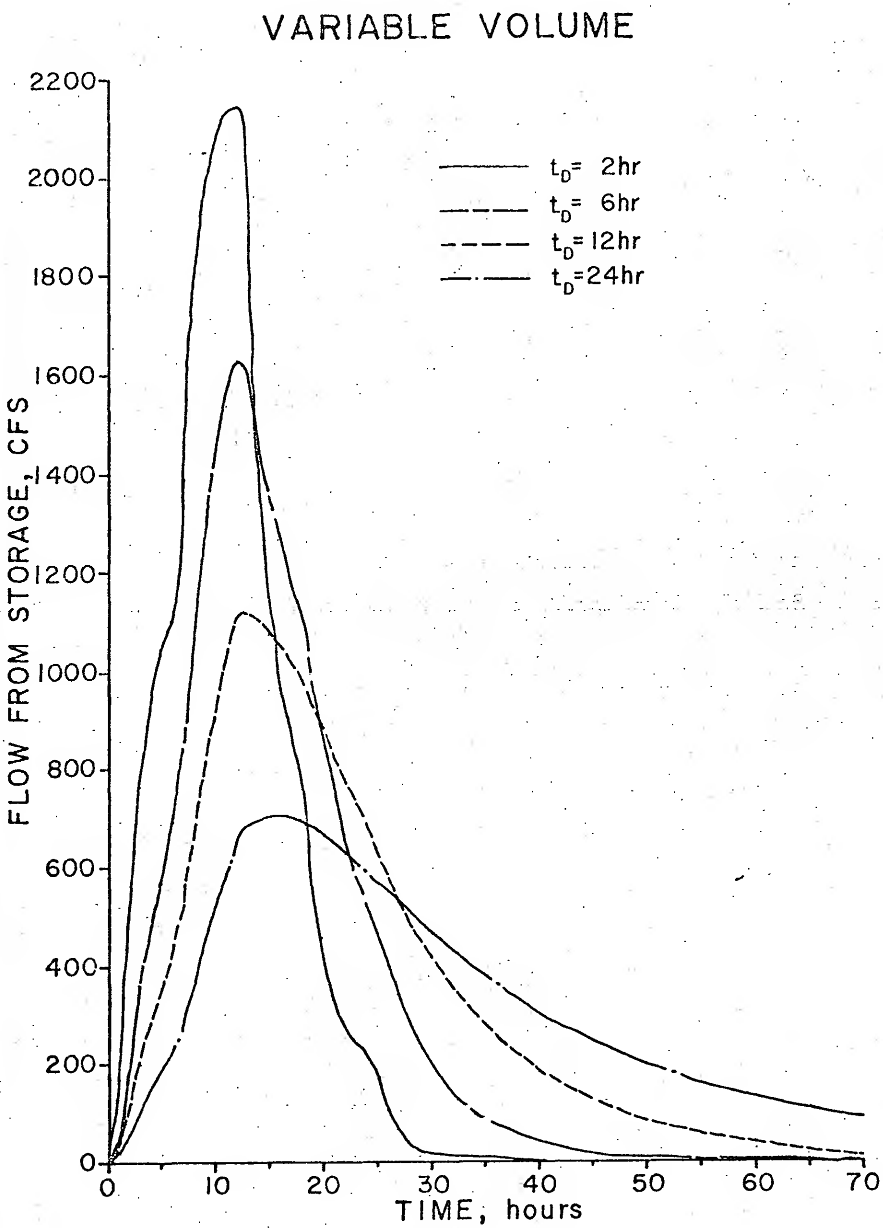


Figure 5-17. Output Flow Rate for Wet Weather Event No. 52 for Varied Detention Time.

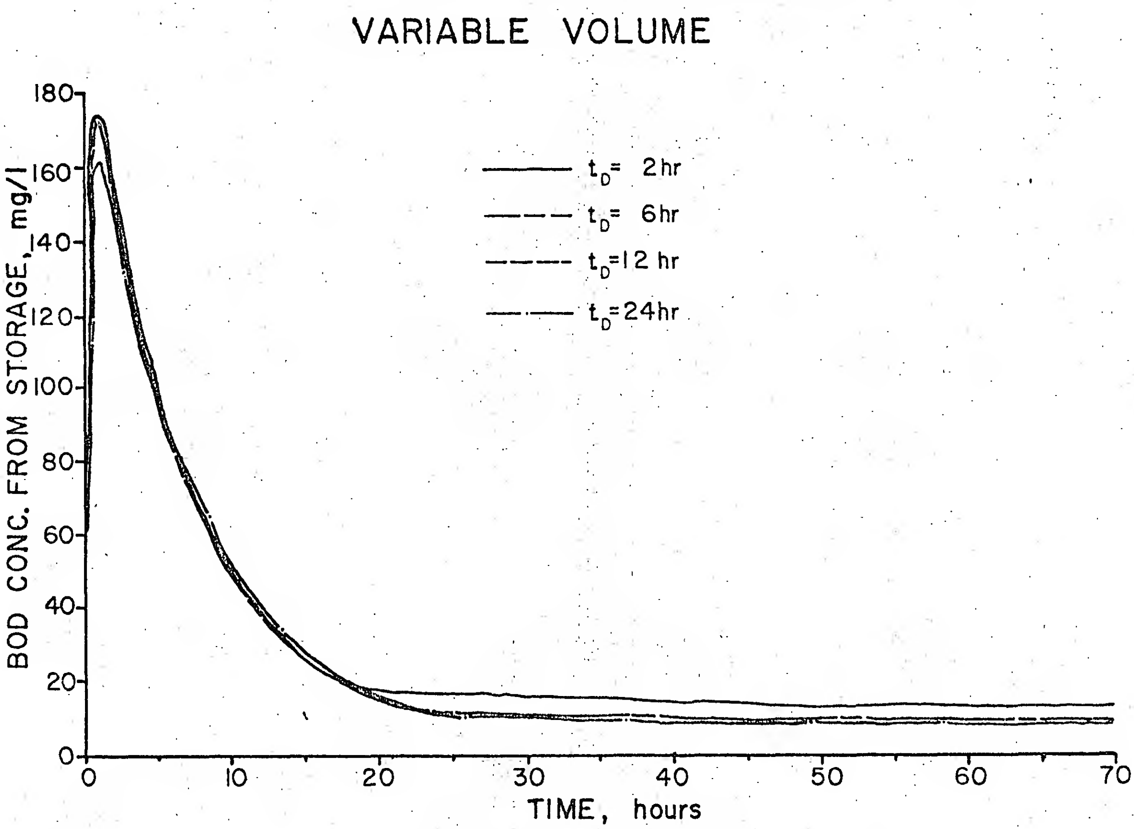


Figure 5-18. BOD Concentration Response for Wet Weather Event No. 52 for Varied Detention Time.

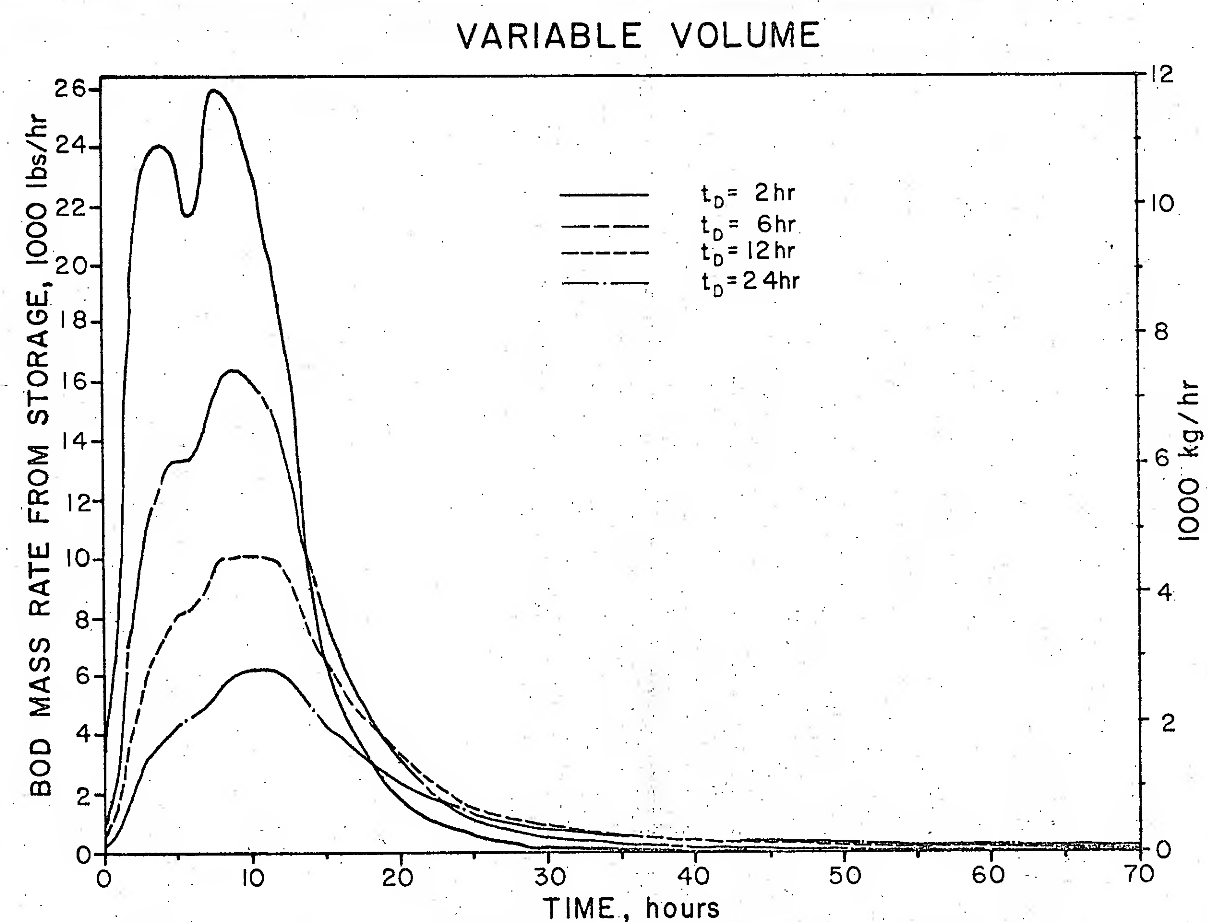


Figure 5-19. BOD Mass Rate Response for Wet Weather Event No. 52 for Varied Detention Time.

Table 5-10

Statistics for Well-Mixed Variable Volume Model for Wet Weather Event No. 52

Item	Input to System	Response from Storage/Treatment			
		t_D = 2 hrs	t_D = 6 hrs	t_D = 12 hrs	t_D = 24 hrs
Peak BOD Concentration	180 mg/l	174	173	172	162
Mean BOD Concentration*	63 mg/l	54	39	29	22
Peak Flow Rate	2267 cfs (64 cu m/sec)	2152 (61)	1636 (46)	1133 (32)	703 (20)
Mean Flow Rate	1070 cfs (30 cu m/sec)	659 (19)	367 (10)	365 (10)	338 (9.6)
Standard Deviation of Flow Rate	771 cfs (22 cu m/sec)	713 (20)	501 (14)	362 (10)	210 (6)
Coefficient of Variation of Flow Rate	0.72	1.08	1.37	0.99	0.62
Peak BOD Mass Rate	38,499 lbs/hr (17,462 kg/hr)	26,026 (11,805)	16,352 (7,417)	10,429 (4,731)	6,119 (2,776)
Mean BOD Mass Rate	15,213 lbs/hr (6,900 kg/hr)	7,975 (3,617)	3,178 (1,442)	2,369 (1,075)	1,673 (759)
Standard Deviation of BOD Mass Rate	14,262 lbs/hr (6,469 kg/hr)	9,883 (4,483)	5,223 (2,369)	3,313 (1,503)	1,839 (834)
Coefficient of Variation of BOD Mass Rate	0.94	1.24	1.64	1.40	1.10

*Flow-weighted average.

Table 5-11

Pollutant Removal Efficiency of
Well-Mixed Variable Volume Model for
Wet Weather Event No. 52

Item	% Reduction for t_D= 2 hrs	t_D=6 hrs	t_D=12 hrs	t_D=24 hrs
Peak BOD Concentration	3.3	3.9	4.4	10.0
Mean BOD Concentration*	14	38	54	65
Peak BOD Mass Rate	32	58	73	84
Mean BOD Mass Rate	48	79	84	89

*From flow-weighted average.

flow-weighted averages of the output BOD concentrations large percentage reductions are noted. An examination of Table 5-11 suggests that excellent reductions of peak and mean BOD mass rates in the discharge are obtained for $t_D \geq 6$ hours. Percentage reductions in peak and mean flow rates are summarized in Table 5-12.

Although peak and mean flow rates are decreased substantially for detention times up to 12 hours, complete flow equalization is not achieved in the strict sense that outflow variability is not reduced. The coefficient of variation of the outflow rate is not improved over the coefficient of variation of the inflow rate except for the 24-hour detention time. Thus, the variable volume system response for a detention time of 48 hours was also investigated. The results are presented in Table 5-13 for Wet Weather Event No. 52 and, as expected, both flow and mass rate equalization are achieved. In addition, significant reductions in peak and mean BOD concentrations are obtained.

Frequency distributions of input BOD concentrations and mass rates for all wet weather events have been presented, respectively, in Figures 5-10 and 5-18 and apply to all system response models. Frequency analyses of output BOD concentrations and mass rates are presented in Figures 5-20 to 5-23. Figure 20 reflects the smoothing of high-concentration values and the increase in frequency

Table 5-12

Reduction in Peak and Mean Flow Rates with Variable Volume Model for Wet Weather Event No. 52

Storage/Treatment System Detention Time	% Reduction in	
	Peak Flow Rate	Mean Flow Rate*
t_D = 2 hrs	5	38
t_D = 6 hrs	28	66
t_D = 12 hrs	50	66
t_D = 24 hrs	69	68

*Averaged over period of discharge, up to 70 hours. If each were averaged over the same overall time period, each would have the same mean discharge.

Table 5-13

Statistics for Variable Volume Model
with Detention Time of 48 Hours*
for Wet Weather Event No. 52

Item	Input to System	Response	% Reduction
Peak BOD Concentration	180 mg/l	134	26
Mean BOD Concentration**	63 mg/l	18	71
Peak Flow Rate	2,267 cfs (64 cu m/sec)	424 (12)	81
Mean Flow Rate	1,070 cfs (30 cu m/sec)	267 (8)	75
Standard Deviation of Flow Rate	771 cfs (22 cu m/sec)	104 (3)	87
Coefficient of Variation of Flow Rate	0.72	0.39	46
Peak BOD Mass Rate	38,499 lbs/hr (17,462 kg/hr)	3,356 (1,522)	91
Mean BOD Mass Rate	15,213 lbs/hr (6,900 kg/hr)	1,095 (497)	93
Standard Deviation of BOD Mass Rate	14,262 lbs/hr (6,469 kg/hr)	926 (420)	94
Coefficient of Variation of BOD Mass Rate	0.94	0.85	10

*From simulation of all wet weather events, the average storage volume used equals 20,729,423 cubic feet (587,057 cu m).

**Flow-weighted average.

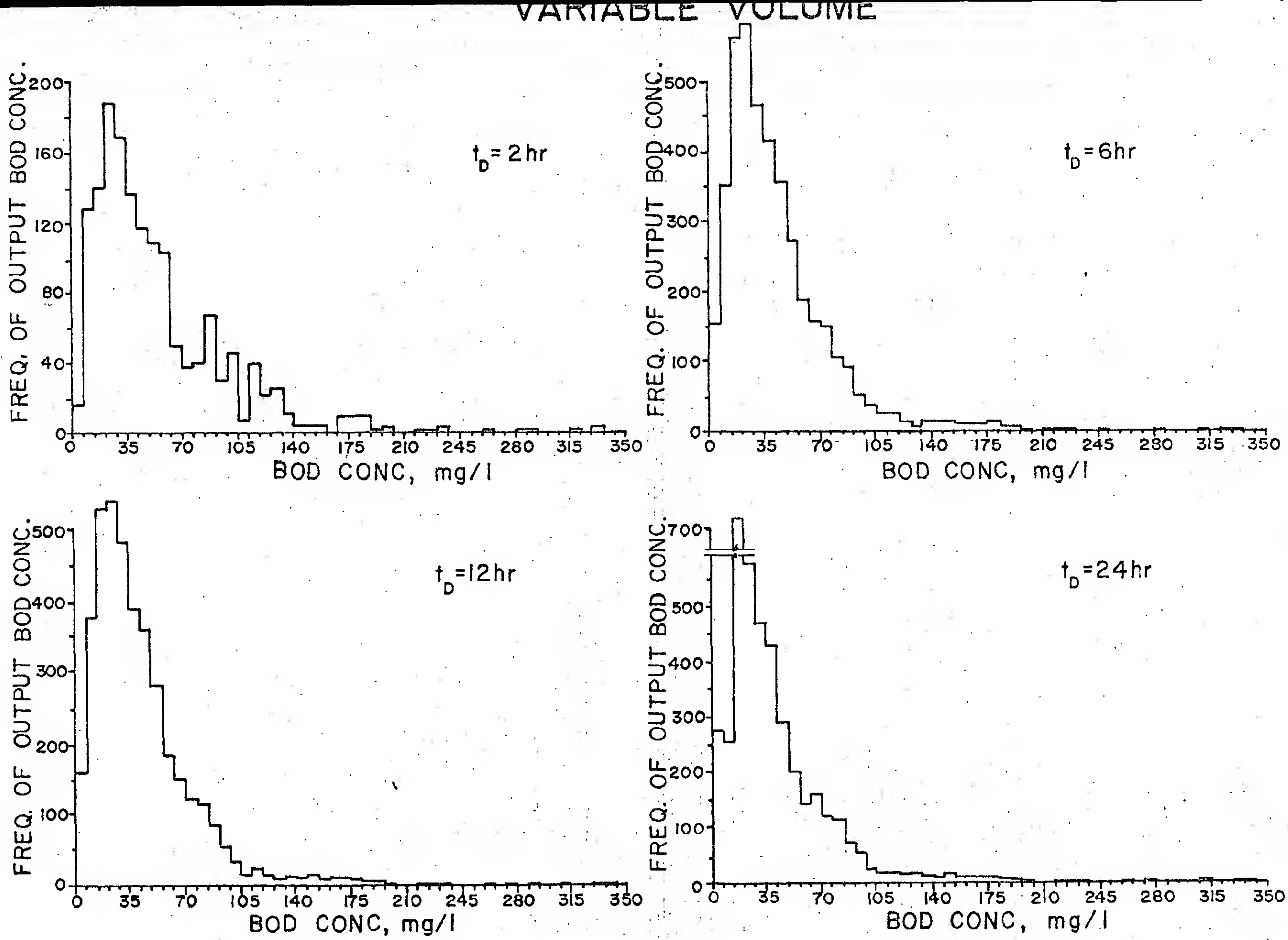


Figure 5-20. Frequency Distribution of Output BOD Concentration for All Wet Weather Events for Variable Volume Storage/Treatment.

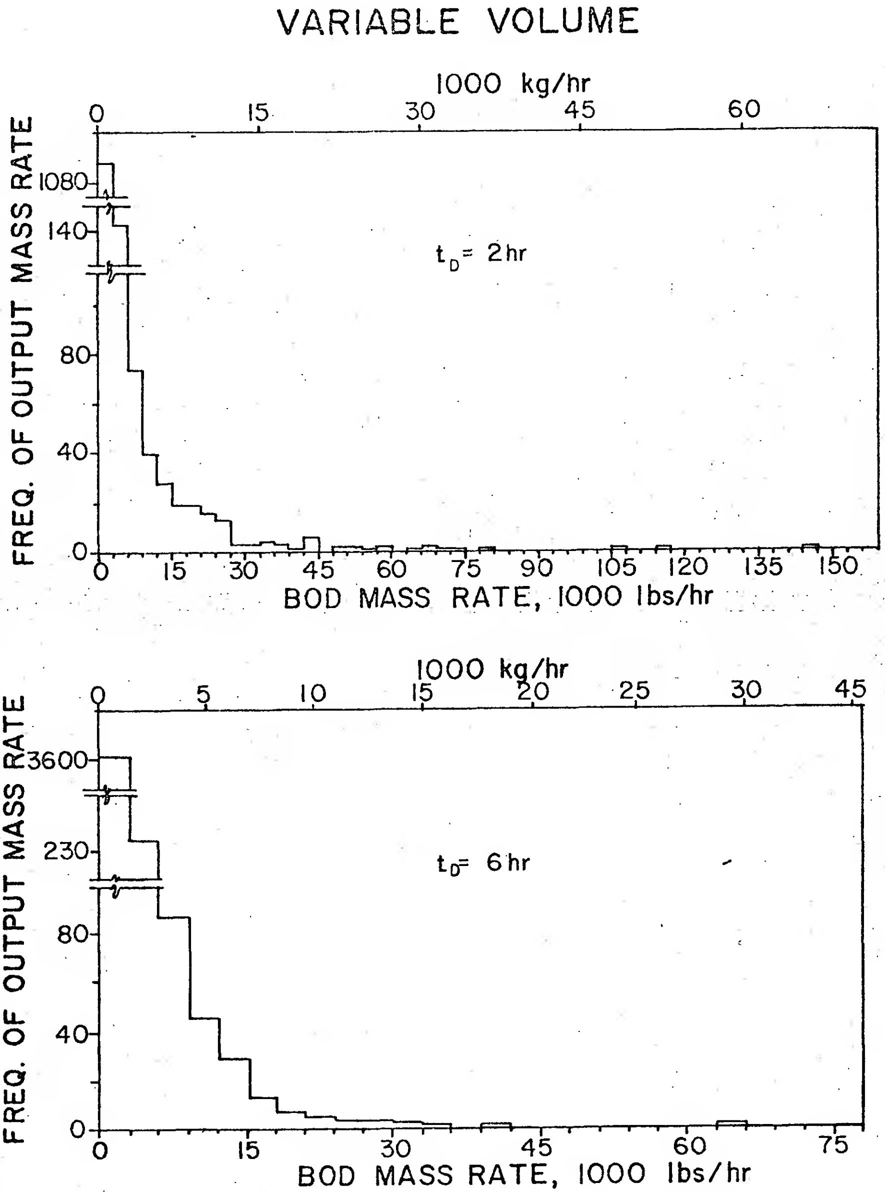


Figure 5-21. Frequency Distribution of Output BOD Mass Rates for All Weather Events for Variable Volume Storage/Treatment.

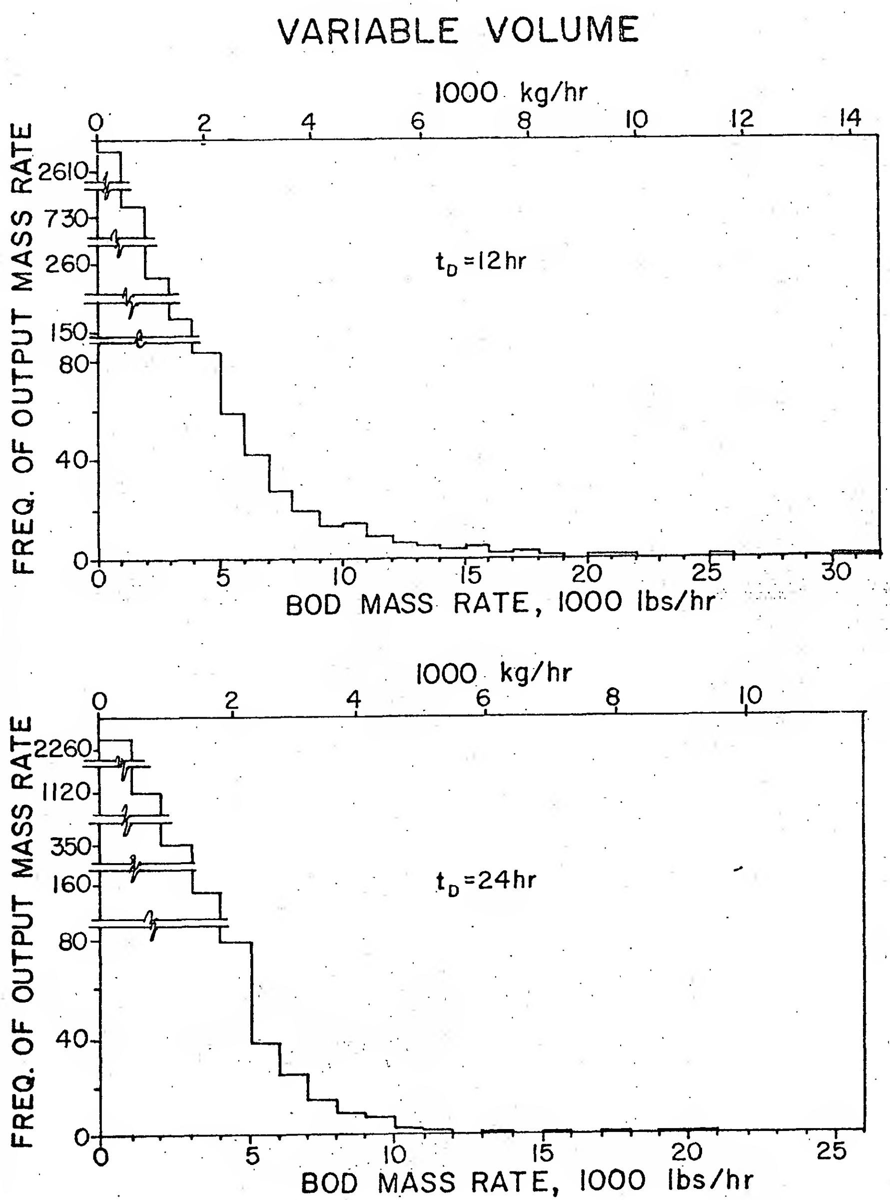


Figure 5-21 continued

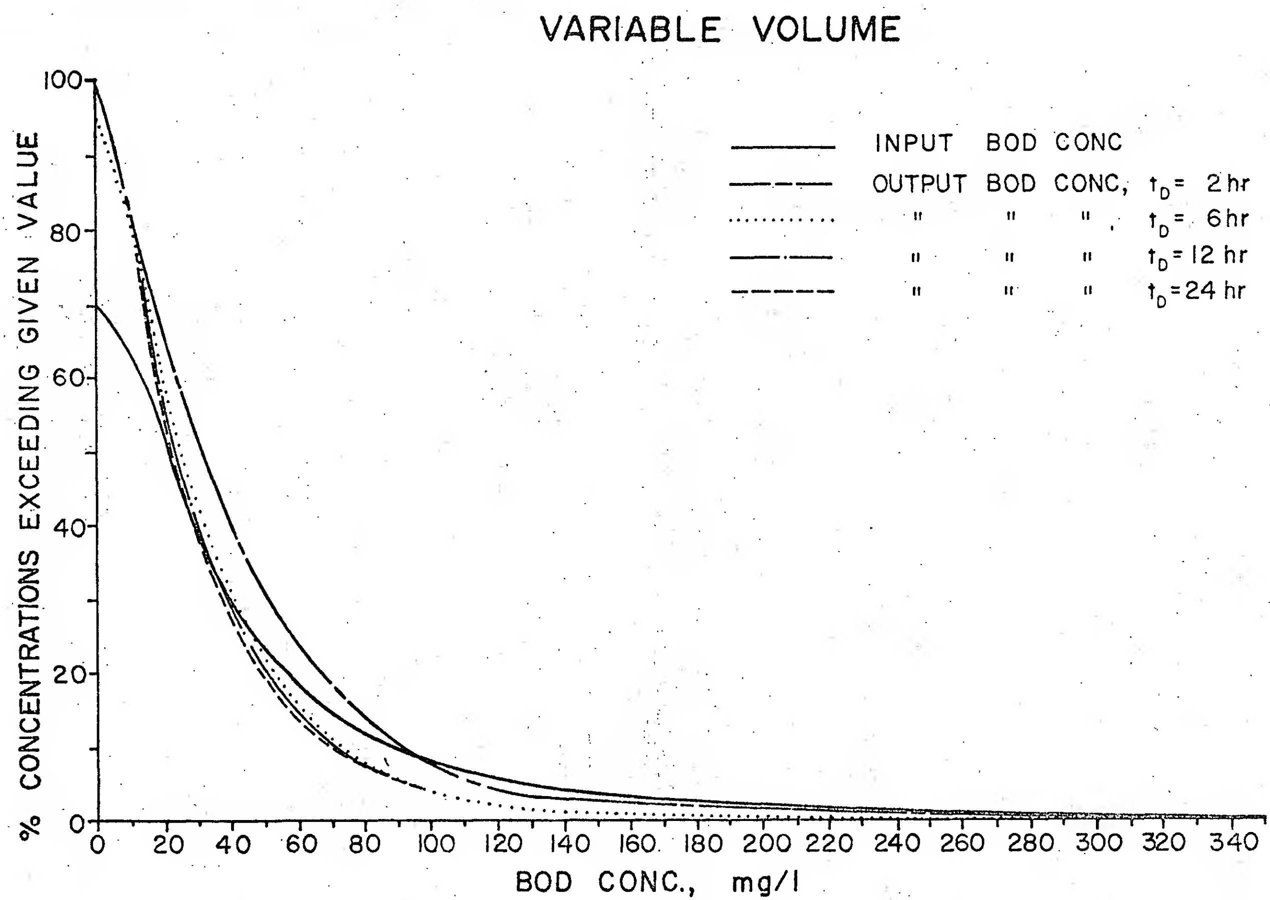


Figure 5-22. Cumulative Frequency of BOD Concentrations for All Wet Weather Events for Variable Volume Storage/Treatment.

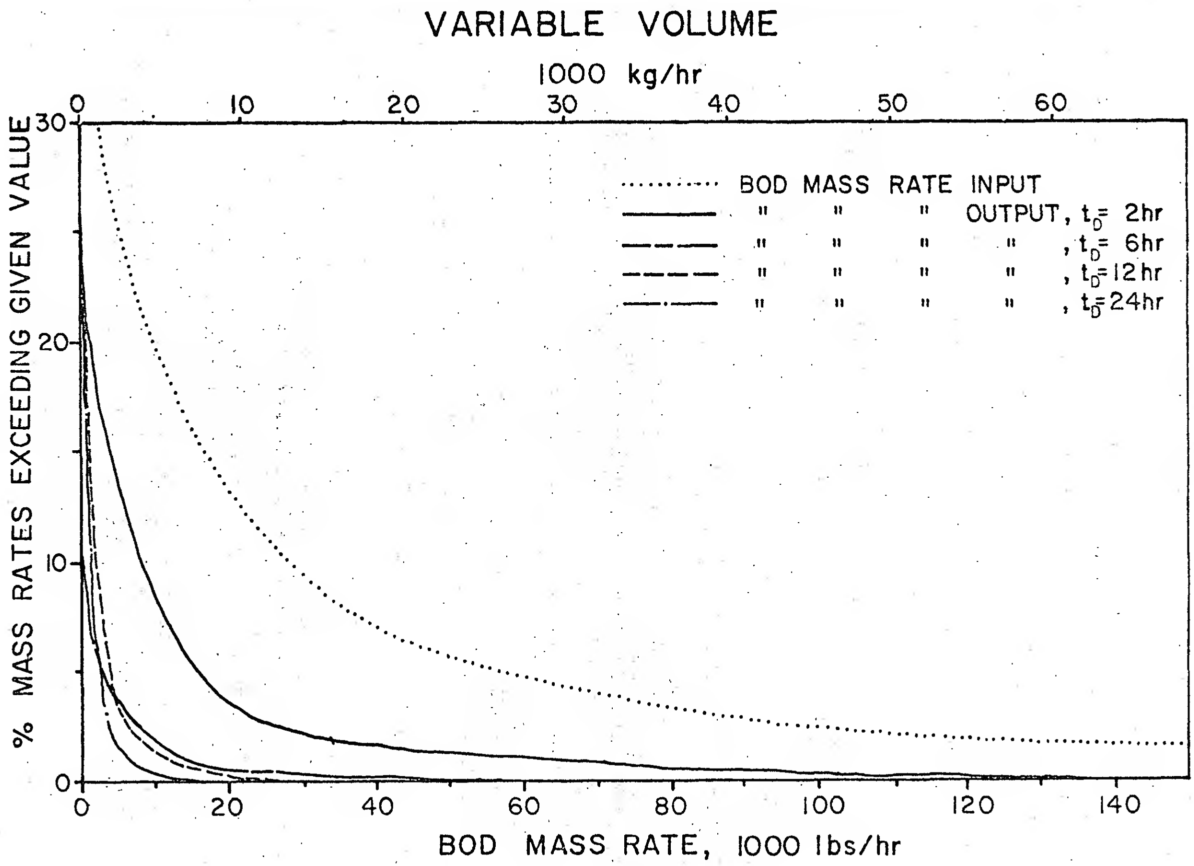


Figure 5-23. Cumulative Frequency of BOD Mass Rates for All Wet Weather Events for Variable Volume Storage/Treatment. Input curve intercepts ordinate at 70%.

of low-range concentrations, as the detention time increases. The effect is significantly more pronounced for BOD mass rates, as shown in Figure 5-21. The cumulative frequency curves for output BOD concentration in Figure 5-22 also reflect the redistribution when compared to the input BOD concentration cumulative frequency curve. The efficiency of the well-mixed variable volume storage/treatment system in reducing BOD mass rates to the receiving waterway is reflected in Figure 5-23. Input and output statistics for all wet weather events are presented in Table 5-14, and the pollutant removal efficiencies are summarized in Table 5-15 and compared to reductions in peak and mean flow rates. Results deviate from those obtained for Wet Weather Event No. 52 in that reductions in peak and mean flow rates appear too high, which in turn explains why such high reductions in BOD mass rates are obtained. First of all, a large number of wet weather events are much shorter in duration than Event No. 52, but flow rates into the storage/treatment system may be much higher. The methodology assumes that all inflows are given the same detention. Thus, if a short event with very large hourly runoff values is processed through the system, it will occupy a basin volume much larger than the averages presented in Table 5-9. Table 5-16 lists the maximum storage used by the largest hourly inflows for each detention time investigated. Thus, if basins are sized according to the average

Table 5-14

Statistics for Well-Mixed Variable Volume Model
for All Wet Weather Events

Item	Input to System	Response from Storage/Treatment			
		t_D = 2 hrs	t_D = 6 hrs	t_D = 12 hrs	t_D = 24 hrs
Peak BOD Concentration	352 mg/l	341	338	337	337
Mean BOD Concentration*	64 mg/l	58	50	46	41
Peak Flow Rate	24,940 cfs (706 cu m/sec)	8,901 (252)	4,412 (125)	2,500 (71)	1,371 (39)
Mean Flow Rate	1,584 cfs (45 cu m/sec)	318 (9)	127 (3.60)	129 (3.65)	129 (3.65)
Standard Deviation of Flow Rate	1,290 cfs (37 cu m/sec)	501 (14)	228 (6.46)	156 (4.42)	92 (2.61)
Coefficient of Variation of Flow Rate	0.81	1.58	1.80	1.21	0.71
Peak BOD Mass Rate	285,563 lbs/hr (129,529 kg/hr)	143,573 (65,124)	65,056 (29,509)	36,571 (16,588)	19,746 (8,957)
Mean BOD Mass Rate	21,717 lbs/hr (9,851 kg/hr)	3,478 (1,578)	1,117 (507)	1,004 (455)	884 (401)
Standard Deviation of BOD Mass Rate	18,250 lbs/hr (8,278 kg/hr)	6,036 (2,738)	2,282 (1,035)	1,436 (65)	808 (367)
Coefficient of Variation of BOD Mass Rate	0.84	1.74	2.04	1.43	0.91

*Flow-weighted average.

Table 5-15

Pollutant Removal Efficiency and Outflow Rate Attenuation of Well-Mixed Variable Volume Model for All Wet Weather Events

Item	% Reduction for Given Detention			
	t_D=2 hrs	t_D=6 hrs	t_D=12 hrs	t_D=24 hrs
Peak BOD Concentration	3	4	4	4
Mean BOD Concentration*	9	22	28	36
Peak BOD Mass Rate	50	77	87	93
Mean BOD Mass Rate	83	94	95	96
Peak Flow Rate	64	82	90	94
Mean Flow Rate	79	92	92	92

*From flow-weighted average.

Table 5-16

Maximum Storage Volume* Required for Desired Detention Time**

Constant Detention Time t_D, Hours	Maximum Basin Volume Used, V_{max} cubic feet (cu m)
2	64,091,360 (1,815,067)
6	95,283,376 (2,698,425)
12	108,058,528 (3,060,218)
24	118,376,288 (3,352,416)

*One storage/treatment variable volume facility for entire urban area.

**As defined by equation (5.51).

volumes, it can be expected that average removals would be lower for the entire record series. In essence, it is likely that these short events, many 1 hour in duration only, are affecting the statistical averages disproportionately.

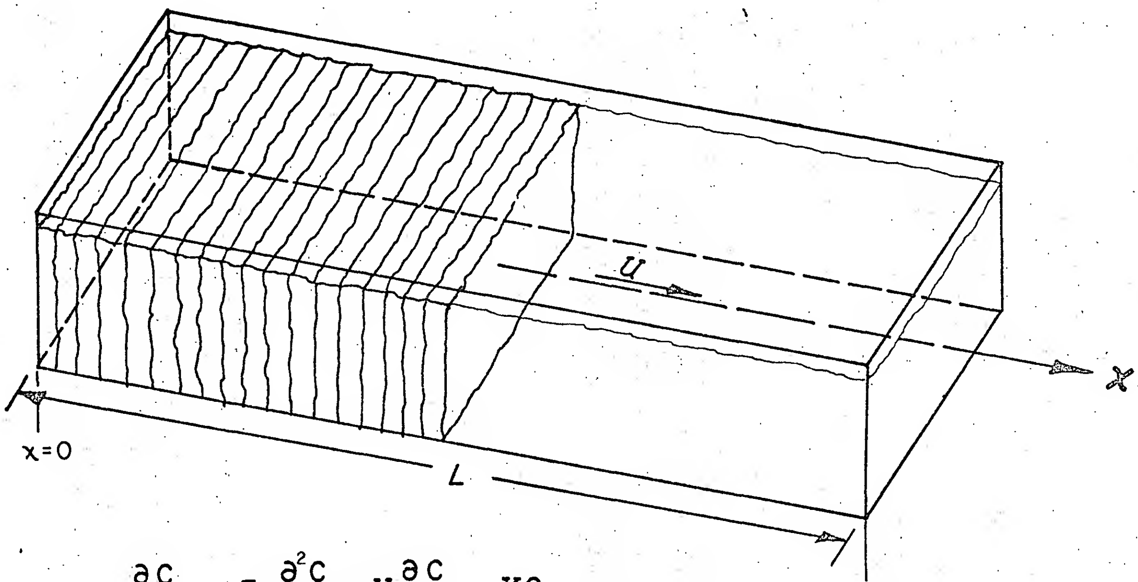
5.4 Dispersive Variable Volume Model

The previous system models studied represent ideal mixing conditions. However, nonideal mixing is a more likely phenomenon where the transport of a pollutant mass in a fluid medium is described by advection and dispersion. Since the detention time in a constant volume system varies according to the influent fluid flow rate, the flow-through velocity is also variable. Most analytical solutions to the convective dispersion equation are for a constant advective velocity (Harleman, 1971). Thus, the model of nonideal mixing is developed for a storage/treatment system represented by a variable volume facility (i.e., constant velocity). The dispersive model may also be applied to a river or an estuary.

Continuously Discharging Plane Source

In Figure 5-24, a plane source perpendicular to the flow axis continuously injects a mass of pollutant into a variable volume storage/treatment system. A

DISPERSIVE VARIABLE VOLUME



$$\frac{\partial C}{\partial t} = E\frac{\partial^2 C}{\partial x^2} - U\frac{\partial C}{\partial x} - KC$$

Figure 5-24. Advection and Dispersion in the Variable Volume Model.

uniform longitudinal velocity advects the pollutant mass in the direction of flow. The governing equation may be derived from the general expression for the one-dimensional convective dispersion equation presented in Chapter II as equation (2.1). For a constant longitudinal dispersion coefficient and first-order decay (sink term), equation (2.1) may be rewritten as

$$\frac{\partial C}{\partial t} = E\,\frac{\partial^2 C}{\partial x^2} - U\,\frac{\partial C}{\partial x} - KC \qquad (5.53)$$

where C = concentration of pollutant in the fluid medium, M/L^3,

$\frac{\partial C}{\partial t}$ = temporal variation in the pollutant concentration, M/L^3T,

E = longitudinal dispersion coefficient, constant along the flow axis, L^2/T,

U = longitudinal velocity in the storage/treatment system, L/T,

$\frac{\partial C}{\partial x}$ = spatial variation in the pollutant concentration, M/L^4, and

K = first-order decay rate of pollutant in the fluid medium, $1/T$.

Equation (5.53) is a linear partial differential equation, parabolic, of the second order, and is formally equivalent to the one-dimensional heat equation (Berg and McGregor, 1966). The fundamental mathematical techniques to solve the Fickian equations of heat conduction

are given rigorous treatment by Carslaw and Jaeger (1959). The most powerful technique applies the Laplace transform to the convective diffusion equation to remove the time variable, leaving an ordinary differential equation of the second order the solution of which yields the transform of the concentration as a function of the space variable x (Crank, 1956). Then, by inverse transformation the solution is interpreted to give an expression for the concentration in terms of both space and time variables, x and t, respectively.

Such a solution is presented by Harleman (1971) for the continuously discharging plane source by a time integration of the instantaneous plane source solution for a constant time rate of injection. Thus,

$$c_2(x,t-\tau) = \int_0^{t_1} \frac{q''(\tau)}{\rho\sqrt{4\,E\cdot(t-\tau)}} \exp\left[-\frac{[x-U\cdot(t-\tau)]^2}{4E\cdot(t-\tau)} + K\cdot(t-\tau)\right] d\tau \qquad (5.54)$$

where $q''(\tau)$ = variable time rate of mass injection per unit area at the plane source, M/L^2T, and

ρ = density of the receiving fluid, M/L^3.

If the time rate of injection per unit area is constant, then the solution to equation (5.54) for any time t

greater than or equal to the duration of injection t_1, is

$$c_2(x,t-t_1) = \frac{q''}{2\rho\Omega} \exp\left[\frac{xU}{2E}\right] \cdot$$

$$\left[\left\{\mathrm{erf}\left(\frac{x+\Omega t}{\sqrt{4Et}}\right) - \mathrm{erf}\left[\frac{x+\Omega\cdot(t-t_1)}{\sqrt{4E\cdot(t-t_1)}}\right]\right\} \exp\left(\frac{x\Omega}{2E}\right)\right.$$

$$\left. -\left\{\mathrm{erf}\left(\frac{x-\Omega t}{\sqrt{4Et}}\right) - \mathrm{erf}\left[\frac{x-\Omega\cdot(t-t_1)}{\sqrt{4E\cdot(t-t_1)}}\right]\right\} \exp\left(-\frac{x\Omega}{2E}\right)\right] \tag{5.55}$$

where $c_2(x,t-t_1)$ = pollutant concentration at distance x along the flow axis and time $t \geq t_1$, mass of pollutant/mass of fluid, dimensionless,

$\Omega = \sqrt{U^2 + 4KE}$, has the dimensions of velocity, L/T, and

t_1 = duration of injection, T.

When the time t is equal to the duration of injection t_1, equation (5.55) reduces to

$$c_2(x,t=t_1) = \frac{q''}{2\rho\Omega} \exp\left[\frac{xU}{2E}\right]$$

$$\left[\left\{\mathrm{erf}\left(\frac{x+\Omega t}{\sqrt{4Et}}\right) - 1\right\} \exp\left(\frac{x\Omega}{2E}\right)\right.$$

$$\left. -\left\{\mathrm{erf}\left(\frac{x-\Omega t}{4Et}\right) - 1\right\} \exp\left(-\frac{x\Omega}{2E}\right)\right] \tag{5.56}$$

for $x > 0$.

Equations (5.55) and (5.56) are the response of the system to one continuous injection for a duration equal to t_1. It is of interest to adapt these solutions to the case where a succession of injections, representing different mass loadings lagged in time, occurs. First, it should be noted that

$$q'' = \frac{\rho Q_1 C_1}{A} \tag{5.57}$$

where Q_1 = influent fluid flow rate, L^3/T,

C_1 = concentration of pollutant in the inflow, dimensionless, and

A = cross-sectional area of the plane source, equal to the width of the storage/treatment system times the depth of flow in the tank, L^2.

Also, since the system has a variable volume and a constant velocity,

$$U = \frac{Q_1(t)}{A(t)} = \frac{L}{t_D} = \text{constant} \tag{5.58}$$

where U = longitudinal velocity, L/T,

$Q_1(t)$ = fluctuating influent fluid flow rate, L^3/T,

$A(t)$ = fluctuating cross-sectional area due to a variable depth, L^2,

L = length of the storage/treatment basin, units of L, and

t_D = dispersive variable volume system detention time, T.

Therefore, it is convenient to define an input function to the system as

$$I(\tau) = \frac{U\ C_1(\tau)}{2\Omega} \tag{5.59}$$

where $I(\tau)$ = system input function, M/L^3, and

$C_1(\tau)$ = concentration forcing function, dimensionless.

From equation (5.55) a system response function may also be defined.

$$f(t-\tau) = \exp\left[\frac{xU}{2E}\right]\left[\left\{\mathrm{erf}\left(\frac{x+\Omega t}{\sqrt{4Et}}\right) - \mathrm{erf}\left[\frac{x+\Omega\cdot(t-\tau)}{\sqrt{4E\cdot(t-\tau)}}\right]\right\}\exp\left(\frac{x\Omega}{2E}\right) - \left\{\mathrm{erf}\left(\frac{x-\Omega t}{\sqrt{4Et}}\right) - \mathrm{erf}\left[\frac{x-\Omega\cdot(t-\tau)}{4E\cdot(t-\tau)}\right]\right\}\exp\left(-\frac{x\Omega}{2E}\right)\right] \tag{5.60}$$

where $f(t-\tau)$ = system step response, dimensionless.

Equation (5.60) may be described as a weighting function, which is defined as the output of the system at any time t to a unit step input ending a time $t-\tau$ before.

Discrete Convolution

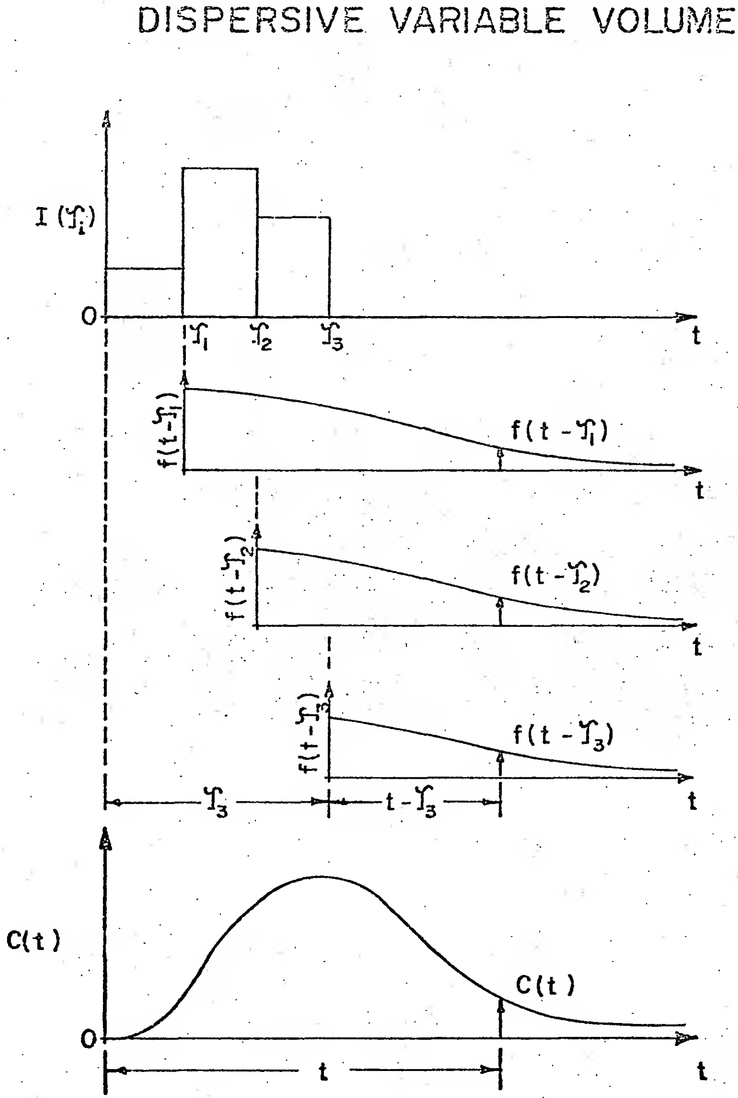
The essence of linearity is the principle of superposition (e.g., Dooge, 1973). If f(x) represents the output to an input x, then the principle is satisfied if for any two inputs x_1, x_2 and constant c,

$$f(x_1 + x_2) = f(x_1) + f(x_2) \tag{5.61a}$$

$$f(cx) = cf(x) \tag{5.61b}$$

Equation (5.61) represents, respectively, the additive and homogeneous properties of a linear system (Bendat and Piersol, 1971). The principle of superposition may be applied to any number of inputs. Figure 5-25 illustrates a summation form of the nature of discrete convolution. The dispersive variable volume storage/treatment system is subjected to a step function input of pollutant concentration, $I(\tau_i)$. Individual step responses to each injection are characterized by $f(t-\tau_i)$. It should be noted that the effects of the various injections are appropriately lagged in time. Thus, the general form of the discrete convolution equation is given by

$$C(t) = \sum_{i=1}^{n} I(\tau_i)f(t - \tau_i) \tag{5.62}$$



$$C(t) = \sum_{i=1}^{n} I(\Upsilon_i)\, f(t-\Upsilon_i)$$

Figure 5-25. Discrete Convolution.

where $C(t)$ = system concentration distribution response to step function inputs, dimensionless,

$i = 1, 2, 3, \ldots, n,$

n = total number of inputs, and

$\tau_i = i\Delta t$, dimensions of time, T.

For computational convenience the step function input is again defined in terms of the discrete forcing function and constant terms, as follows:

$$I(\tau_i) = \frac{U\ C_1(\tau_i)}{2\Omega} \tag{5.63}$$

where $C_1(\tau_i)$ = discrete concentration forcing function, dimensionless.

Since the system response is of interest only at the storage/treatment unit's outfall, then equation (5.60) is evaluated for $x = L$. To obtain adequate computational accuracy, it is necessary to solve equation (5.60) over a wide range of times prior to its invocation in the convolution equation. Any given storm may be approximated as a step function consisting of n segments of different pollutant injections $I(\tau_i)$, each of the same duration t_1 (which is equal to the time step length, Δt). The system step response, equation (5.60), is evaluated at short time increments Δt_2, and an average is obtained over the time step, Δt.

$$f(t_{ij} - t_1) = \frac{1}{m} \left[\frac{1}{2} f\,(t_j - t_1) + \ldots\right.$$

$$\left. f(t_j + i\,\Delta t_2 - t_1) \ldots + \frac{1}{2} f(t_j + m\,\Delta t_2 - t_1)\right] \qquad (5.64)$$

where t_j = any time $t \geq t_1$,

$i = 1, 2, \ldots, m-1,$

$m = \frac{\Delta t}{\Delta t_2}$ = total number of increments desired per time step,

$j = 1, 2, \ldots, n, \ldots, N,$ and

N = any desired upper bound.

Thus, a large array of values of the function $f(t_{ij} - t_1)$ is stored in the digital computer's memory for subsequent recall by equation (5.62). Values of t_i up to 100 hours are evaluated, using a computational increment $\Delta t_2 = 0.2$ hours in equation (5.64). It should be noted that equation (5.62) provides the system response, at each time step, due to several inputs lagged accordingly.

Solutions are extended well beyond the time of the last injection of urban runoff pollutants for two important reasons. First, the transient effects may still be substantial several hours after the storm event has ended. Second, solutions to the convective dispersion equation are limited in their inability to handle initial conditions of concentration in the storage/treatment system (prior to injection). The fluid flow

rate computations presented for the well-mixed variable volume model are applicable to the dispersive variable volume model, including interevent outflow. The dispersive variable volume system mass rate response is simply obtained, for each time step, by

$$W(t) = C(t) \cdot \overline{Q}_2(t) \tag{5.65}$$

where $C(t)$ is defined by equation (5.62) and $\overline{Q}_2(t)$ by equation (5.37).

Model Application

The mathematical models representing the well-mixed storage/treatment systems did not require knowledge of specific physical dimensions, such as length, width and depth. In their application to a real urban site, the models were able to predict storage volumes from known residence times and inflows. The system length is required to apply the dispersive variable volume model. From an established length and a fixed detention time, a constant longitudinal velocity is obtained.

In sewage treatment plant design practices, the length of sedimentation tanks is limited to a maximum of 300 feet (91.5 m) for rectangular shapes (ASCE, 1959). A survey of 42 plants indicated the depth of tanks ranged from 7.5 to 16.5 feet (2.3 to 5.0 m), and the

length-to-width ratio averaged about 4.0/1 (ASCE, 1959). The City of Milwaukee evaluated the merits of detention tanks as a practical method for abatement of combined sewer overflow from a 570-acre (231-ha) drainage area (City of Milwaukee et al., 1975). A project detention tank was constructed which measured 420 feet (128 m) in length, 75 feet (23 m) in width, and had a depth of 16 feet (4.9 m). The pollutant removal effectiveness of this facility is discussed in the summary at the end of this chapter.

From these considerations, the length of the dispersive variable volume system was fixed at 300 feet (91.5 m). From storage volumes reported in Tables 5-9 and 5-16, width and depth requirements may be determined according to the dictates of good design practice, engineering judgment, and experience. It is recognized that in an actual implementation several detention tanks would be required for practical reasons. However, the length assumed in this analysis is realistic and, consequently, the results obtained from the digital computer simulations should prove to be quite helpful in selecting adequate levels of detention for many applications.

The selection of a longitudinal dispersion coefficient for the storage/treatment system proved to be more difficult. Measurements of this important parameter were not a part of the data collection effort in the

Milwaukee study. Murphy and Timpany (1967) reported an average axial dispersion coefficient of 5,730 ft^2/hr (532 sq. m/hr) in an activated sludge aeration tank 66 feet (20 m) long, 30 feet (9 m) wide, and 15 feet (4.6 m) deep. The tank was subjected to constant air-flow from sparger-type air diffusers inducing spiral mixing. Thirumurthi (1969) suggested that a dimensionless dispersion number ($t_D E/L^2$) of 0.25 was reasonable for a waste stabilization pond 65 feet (19.8 m) long and with a 3-day detention time. This would yield an approximate dispersion coefficient of 15 ft^2/hr (1.4 sq. m/hr). Recently, Novotny and Stein (1976) reported a longitudinal dispersion coefficient of 68 ft^2/hr (6.3 sq. m/hr) in a relatively well-functioning equalization basin at a batch organic chemical wastewater treatment plant. The basin had a volume of 133,648 cubic feet (3,785 m^3) and an average inflow of 1.22 cfs (0.03 m^3/sec or 2,980 m^3/day). Thus, the average detention time provided was 30 hours, and since no mixing devices were present, equalization by dispersion prevailed.

Intuitively, a forced aeration tank should have a much higher dispersion coefficient than a storage/ treatment facility with no mixing devices. In fact, this is demonstrated by the experimental studies of Murphy and Timpany (1967), where the dispersion coefficient was found to increase as a logarithmic function of

the air flow rate. Thus, values near the Novotny and Stein (1976) measurement should be expected for storage/treatment systems characterized by nonideal mixing. A series of trials was conducted by solving the system response function, equation (5.59), for various dispersion coefficients and the length selected above. Solutions were obtained with a dispersion coefficient of 72 ft^2/hr (6.7 sq. m/hr) for detention times of 6, 12, and 24 hours. However, a minimum value of 192 ft^2/hr (17.8 sq. m/hr) was required for solution at $t_D = 2$ hours because of numerical problems resulting from actual evaluation of equation (5.60) on the digital computer.

Discrete inputs of flow, BOD concentration and BOD mass rate were presented earlier in Figures 5-4, 5-5, and 5-6 for a single storm event having a 24-hour runoff duration (Wet Weather Event No. 52). The response of the dispersive variable volume model to these inputs is shown in Figures 5-26 to 5-28. The output hydrographs presented in Figure 5-26 are the same as those predicted by the well-mixed variable volume model. As stated previously, the same flow balance equations were applied to both the well-mixed and the dispersive cases. Figures 5-27 and 5-28 illustrate the transient response to BOD inputs. In general, substantial reductions in the peak BOD concentrations and mass rates are visible as the residence time of inflows is increased. An

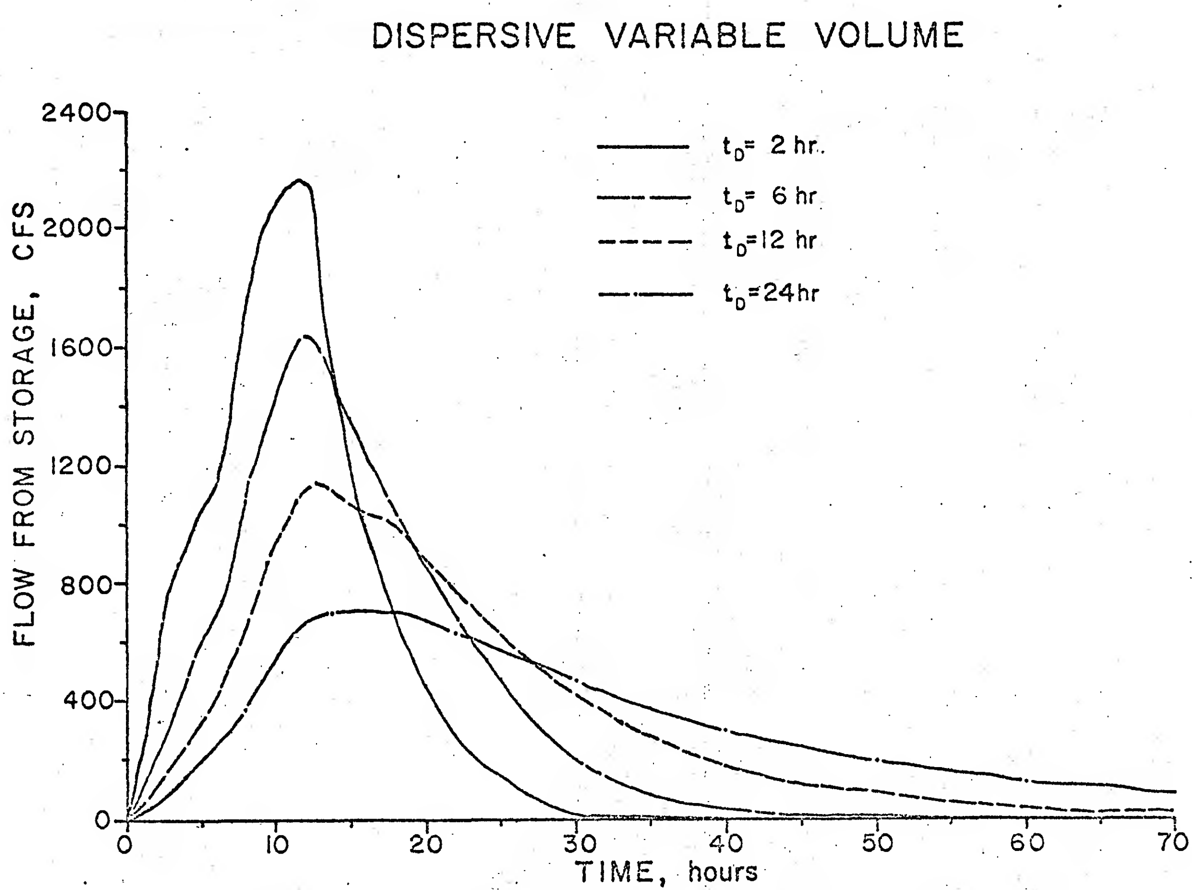


Figure 5-26. Flow Rate Response for Wet Weather Event No. 52.

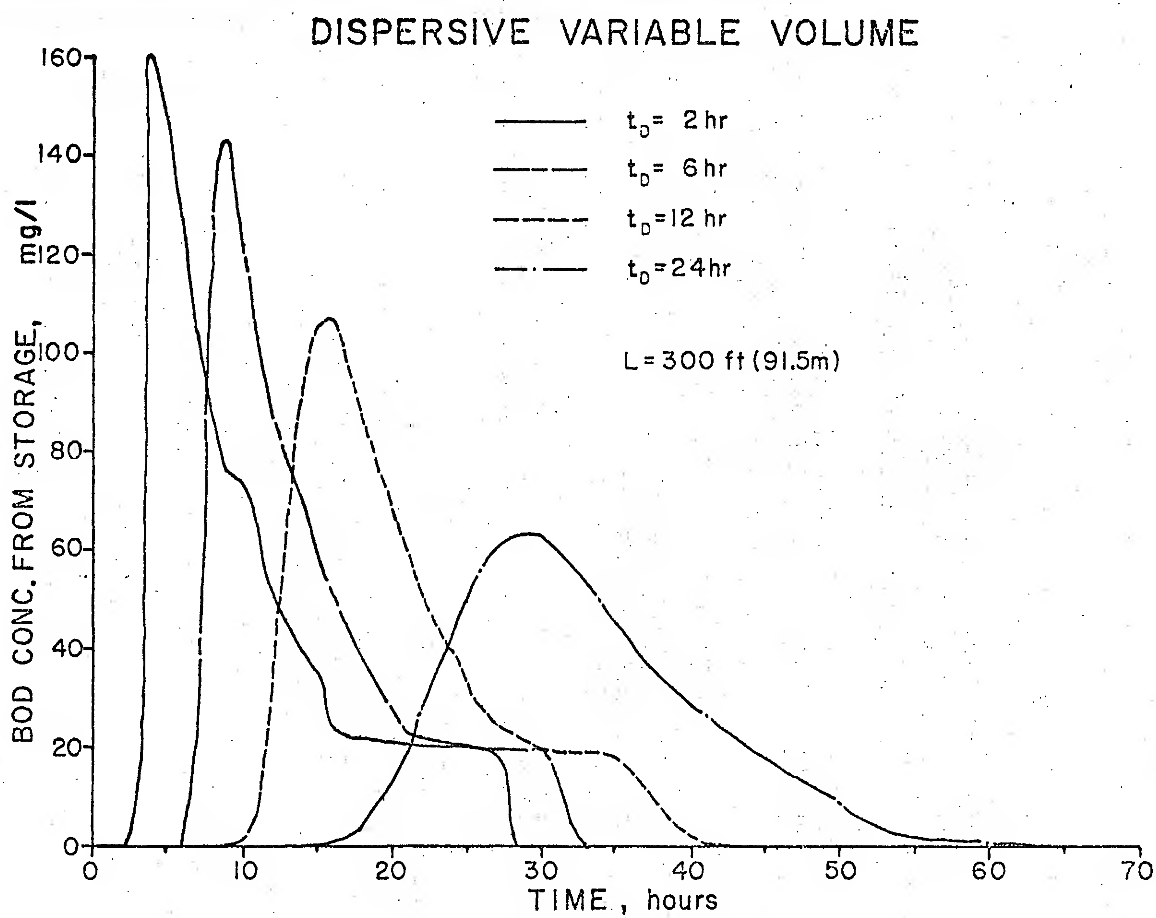


Figure 5-27. BOD Concentration Transient Response for Wet Weather Event No. 52.

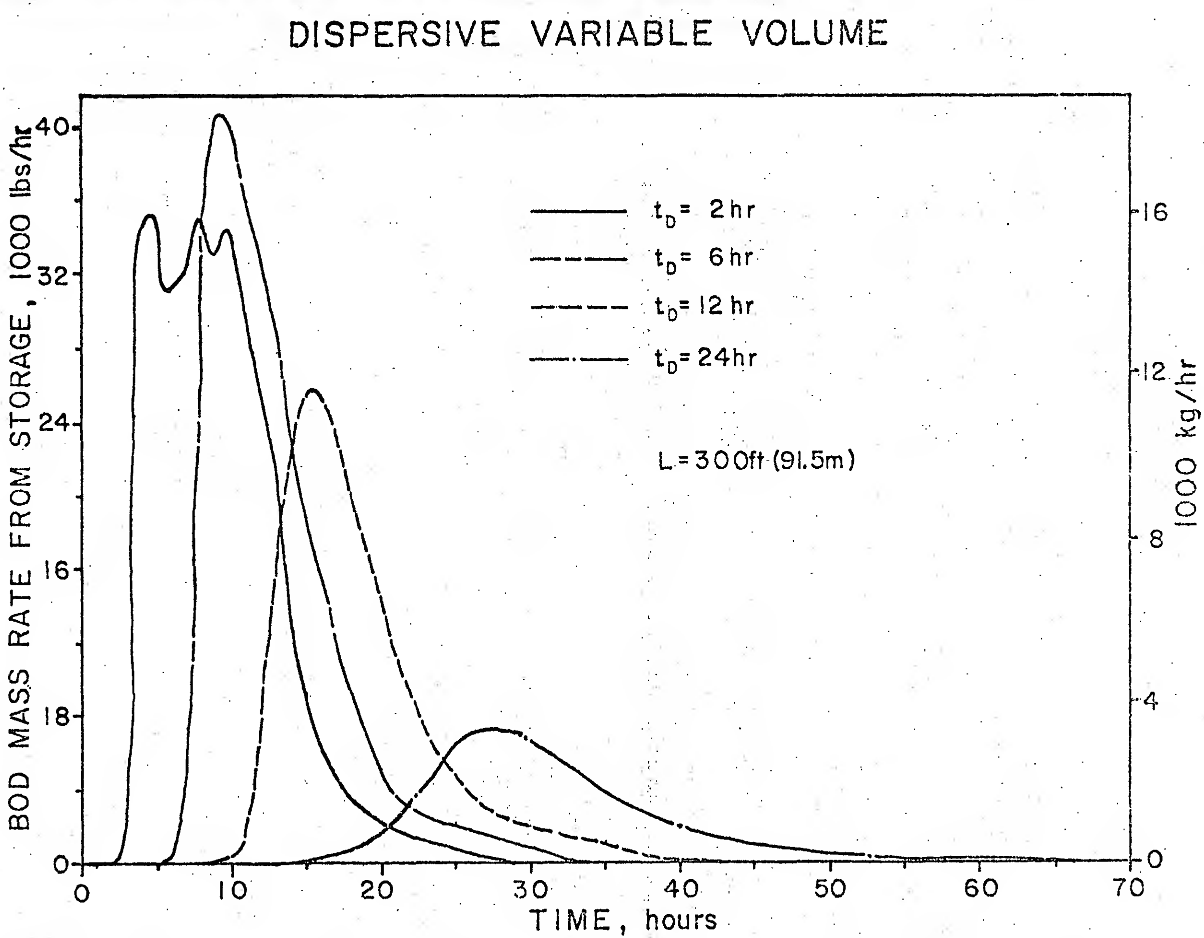


Figure 5-28. BOD Mass Rate Transient Response for Wet Weather Event No. 52.

exception occurs at a detention time of 6 hours. The corresponding outflow hydrograph segments (rising limb, peak, and receding limb) in Figure 5-26 almost coincide temporally with those of the BOD concentration polluto-graph, Figure 5-27. Basin dynamics are such that the peak BOD concentration occurs when outflow from the basin is near its peak discharge. Thus, the peak BOD mass rate for t_D = 6 hours is slightly higher than that at t_D = 2 hour.

Input and output statistics for BOD concentrations and mass rates are presented in Table 5-17 for the single storm event. Pollutant removal efficiencies for peak and mean BOD concentrations and mass rates are summarized in Table 5-18. From these tables it appears that adequate pollutant removal is achieved only at detention times of 12 hours or higher.

Frequency distributions of BOD inputs for all events have been presented in Figures 5-10 and 5-13. Frequency distributions of the output BOD concentrations and BOD mass rates are shown, respectively, in Figures 5-29 and 5-30. It is clear from Figure 5-29 that there is a shift in the frequency distribution towards the lower range BOD concentrations as detention time increases. The same effect is observed in Figure 5-30 for output BOD mass rates, although apparently more pronounced. Cumulative frequency curves for BOD concentrations and

Table 5-17

Statistics for Dispersive Variable Volume Model
for Wet Weather Event No. 52

Item	Input to System	Response from Storage/Treatment System**			
		t_D=2 hrs	t_D=6 hrs	t_D=12 hrs	t_D=24 hrs
Peak BOD Concentration	180 mg/l	161	143	107	63
Mean BOD Concentration*	63 mg/l	63	62	50	27
Peak BOD Mass Rate	38,499 lbs/hr (17,462 kg/hr)	34,610 (15,699)	40,229 (18,248)	25,297 (11,475)	7108 (3224)
Mean BOD Mass Rate	15,213 lbs/hr (6900 kg/hr)	13,700 (6214)	11,635 (5278)	6999 (3175)	2216 (1005)
Std. Dev. of BOD Mass Rate	14,262 lbs/hr (6469 kg/hr)	14,203 (6442)	13,447 (6099)	8239 (3737)	2429 (1102)
Coefficient of Variation of BOD Mass Rate	0.94	1.04	1.16	1.18	1.10

*Flow-weighed average.

**E = 72 ft^2/hr (6.7 sq m/hr) for t_D = 6, 12, 24 hrs and
E = 192 ft^2/hr (17.8 sq m/hr) for t_D = 2 hrs.

Table 5-18

Pollutant Removal Efficiency of
Dispersive Variable Volume Model
for Wet Weather Event No. 52

Item	% Reduction for t_D=2 hrs	t_D=6 hrs	t_D=12 hrs	t_D=24 hrs
Peak BOD Concentration	11	21	41	65
Mean Bod Concentration*	0	2	21	57
Peak BOD Mass Rate	10	-4**	34	82
Mean BOD Mass Rate	10	24	54	85

*From flow-weighted average.

**Slight increase in peak mass rate because the peak BOD concentration occurs when outflow from the system is near its highest discharge.

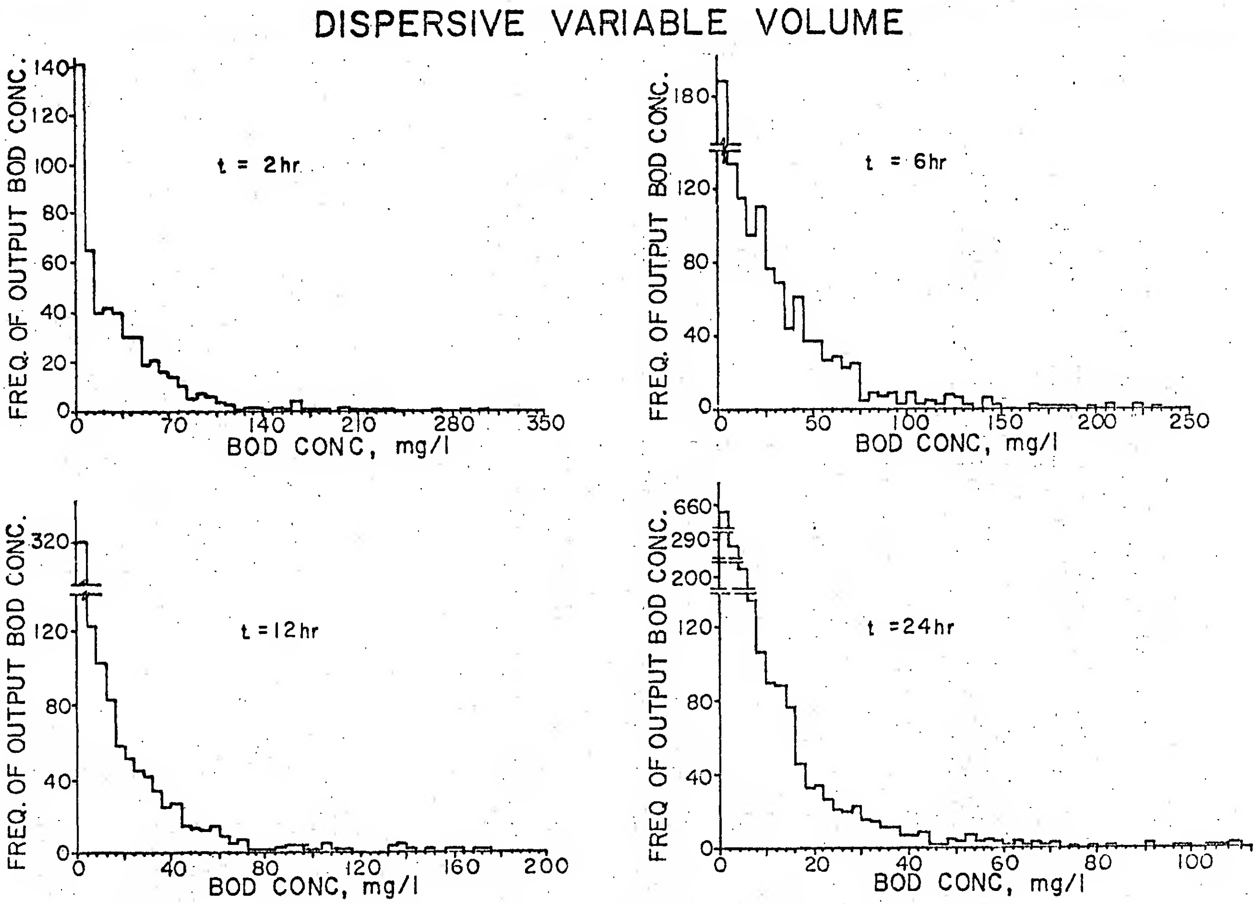


Figure 5-29. Frequency Distribution of Output BOD Concentrations for All Wet Weather Events for Dispersive Variable Volume Model.

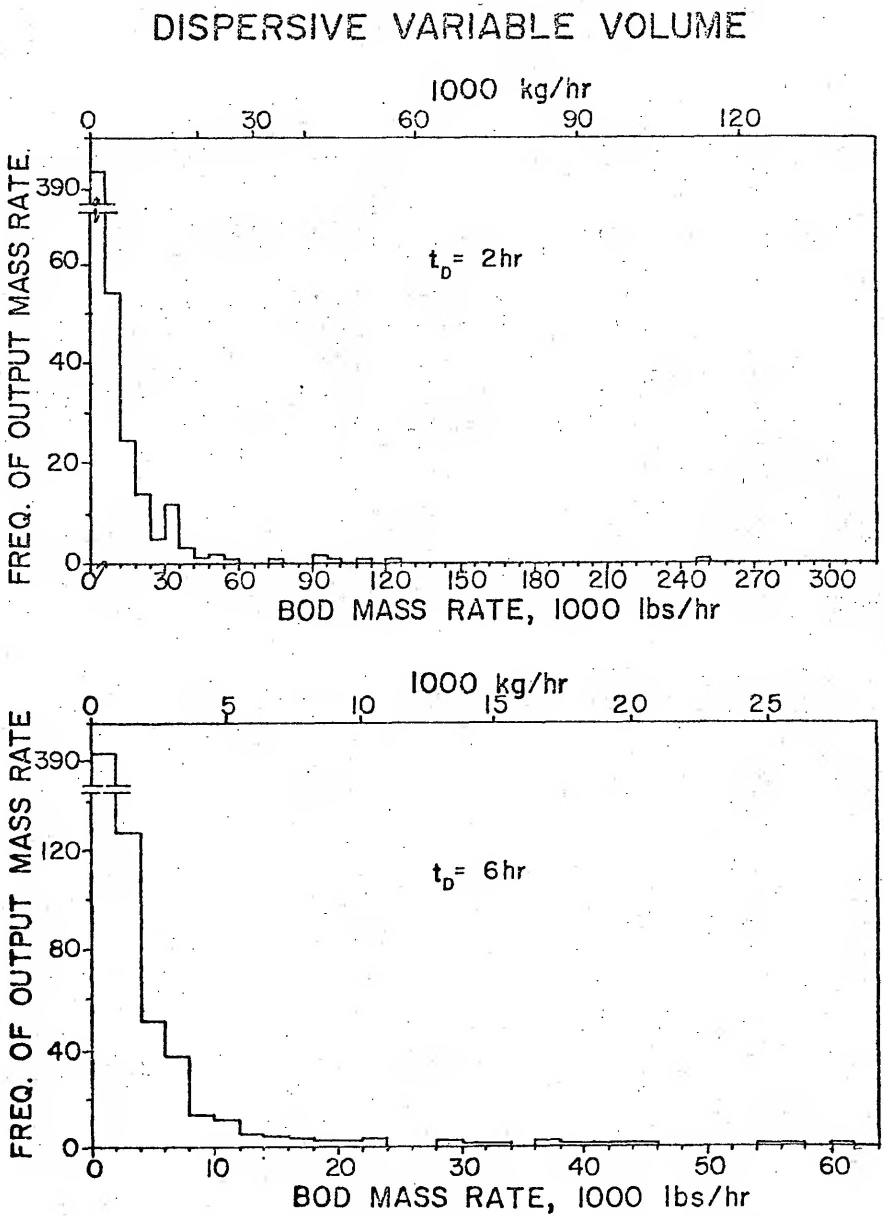


Figure 5-30. Frequency Distribution of Output BOD Mass Rates for All Wet Weather Events for Dispersive Variable Volume Model.

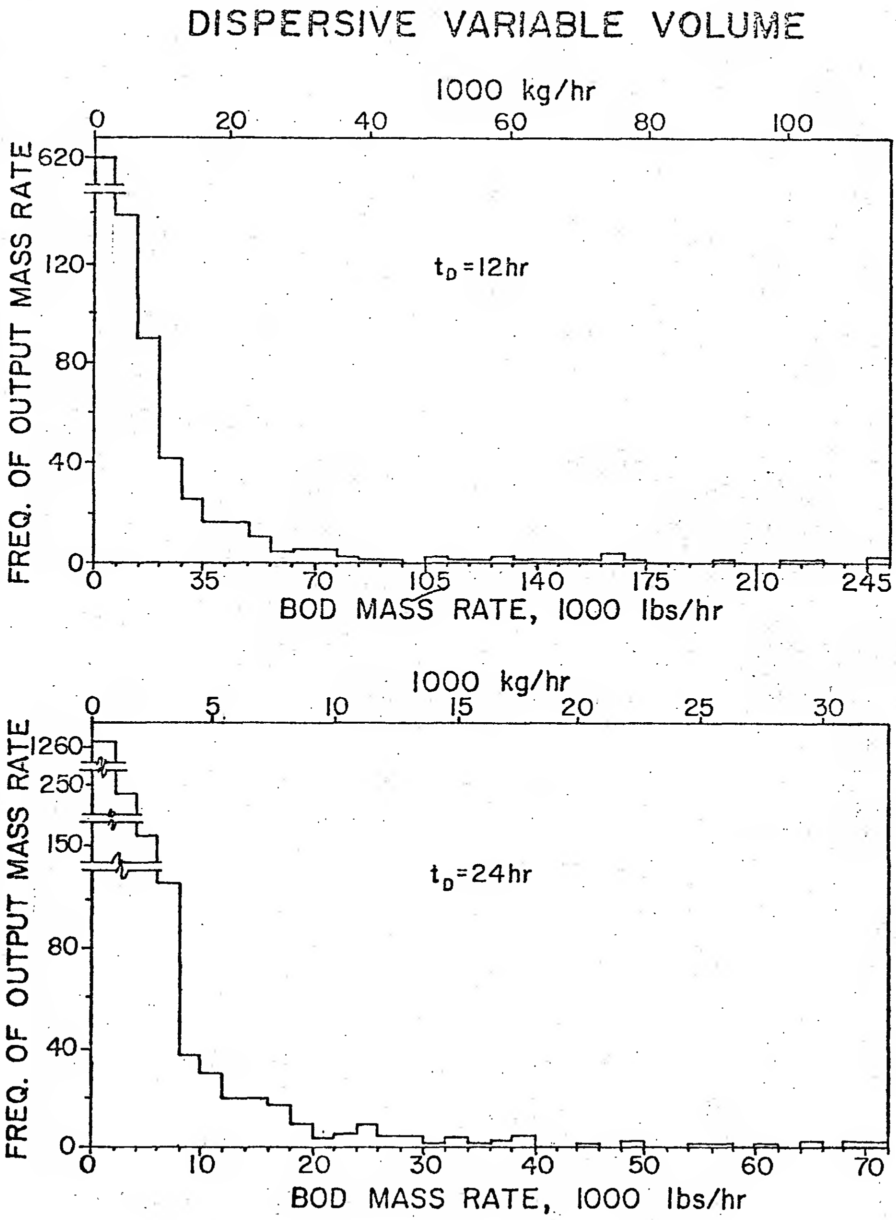


Figure 5-30 continued

mass rates are plotted, respectively, in Figures 5-31 and 5-32. When compared to the input BOD concentration cumulative frequency curve, it is clear in Figure 5-31 that substantial incremental reductions in output BOD concentrations are achieved for each increase of the system detention time. These reductions are even more drastic with respect to BOD mass rates as shown in Figure 5-32.

Input and output statistics for all wet weather events are presented in Table 5-19, and the pollutant removal efficiencies for peak and mean BOD concentrations and mass rates are summarized in Table 5-20. Since peak pollutant loadings are important factors in receiving water quality, the performance of the dispersive variable volume storage/treatment system appears to be quite adequate for detention times greater than or equal to 6 hours. As with the well-mixed variable volume model, percentage reductions of BOD mass rates are quite high, due to outflow rate attenuation (see Tables 5-15 and 5-16).

It is important to examine the sensitivity of the dispersive variable volume model to the magnitude of the longitudinal dispersion coefficient, E. For BOD inputs from a single storm event (see Figure 5-5) and constant t_D, K, and L, the BOD concentration response for different dispersion coefficients is illustrated in

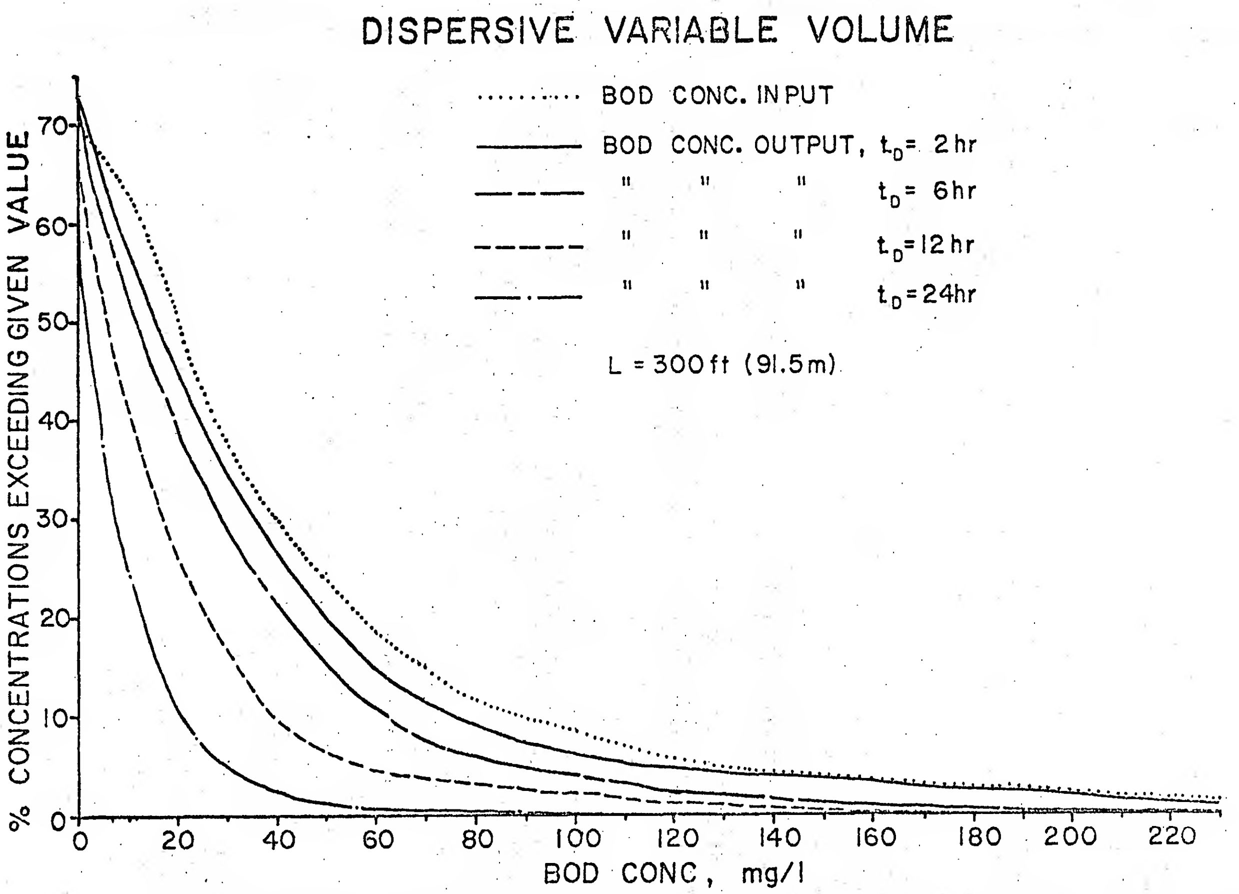


Figure 5-31. Cumulative Frequency of BOD Concentrations for All Wet Weather Events for Dispersive Variable Volume Model.

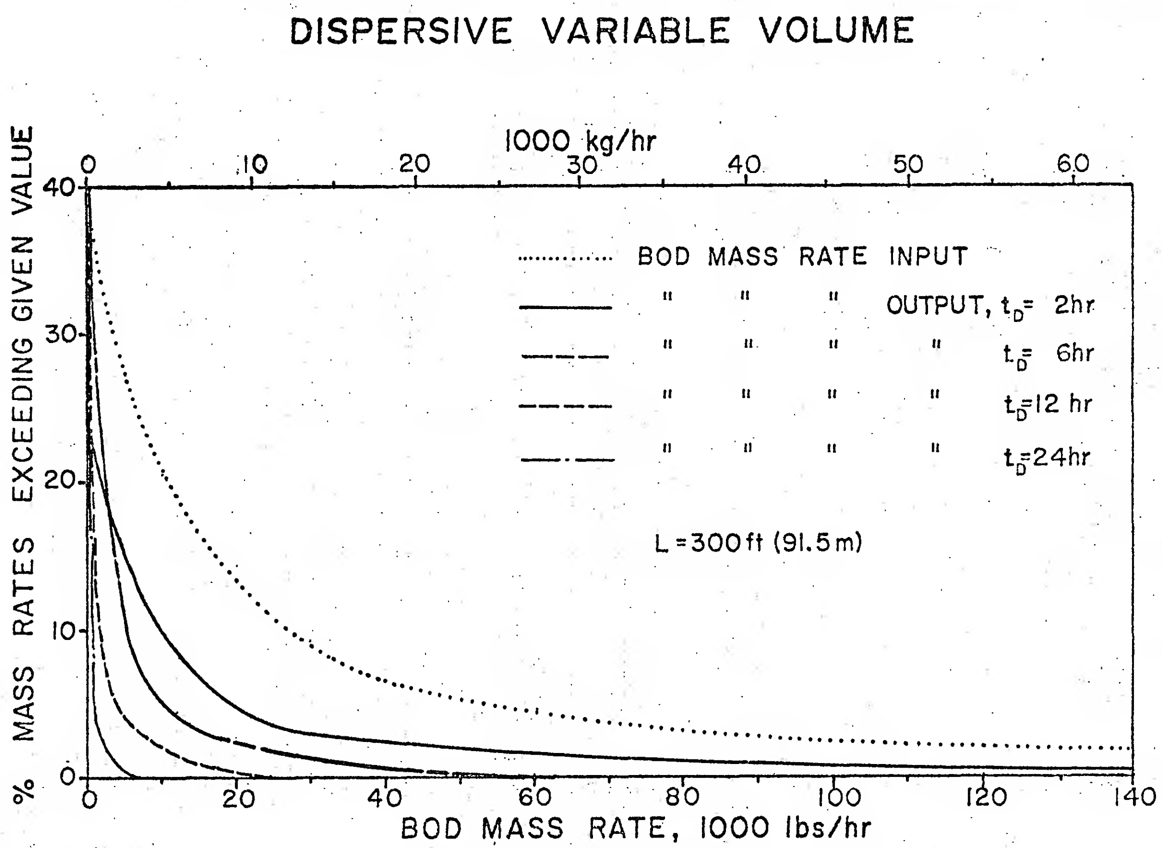


Figure 5-32. Cumulative Frequency of BOD Mass Rates for All Wet Weather Events for Dispersive Variable Volume Model.

Table 5-19

Statistics for Dispersive Variable Volume Model for All Wet Weather Events

Item	Input to System	Response from Storage/Treatment System**			
		t_D=2 hrs	t_D=6 hrs	t_D=12 hrs	t_D=24 hrs
Peak BOD Concentration	352 mg/l	306	234	177	106
Mean BOD Concentration*	64 mg/l	40	28	18	8
Peak BOD Mass Rate	285,563 lbs/hr (129,529 kg/hr)	250,400 (113,580)	59,815 (27,132)	25,297 (11,475)	7108 (3224)
Mean BOD Mass Rate	21,717 lbs/hr (9851 kg/hr)	5955 (2701)	2418 (1097)	945 (429)	257 (117)
Std. Dev. of BOD Mass Rate	18,250 lbs/hr (8278 kg/hr)	7931 (3597)	2822 (1280)	1003 (455)	258 (117)
Coefficient of Variation of BOD Mass Rate	0.84	1.33	1.17	1.06	1.00

*Flow-weighted average.

**E = 72 ft^2/hr (6.7 sq m/hr) for t_D = 6, 12, 24 hrs and E = 192 ft^2/hr (17.8 sq m/hr) for t_D = 2 hrs.

Table 5-20

Pollutant Removal Efficiency of
Dispersive Variable Volume Model for
All Wet Weather Events

Item	% Reduction for t_D=2 hrs	t_D=6 hrs	t_D=12 hrs	t_D=24 hrs
Peak BOD Concentration	13	34	50	70
Mean BOD Concentration*	37	56	72	87
Peak BOD Mass Rate	12	79	91	97
Mean BOD Mass Rate	72	89	95	98

*Flow-weighted average.

Figure 5-33. The results underscore the need for choosing this parameter carefully when actual measurements are not available. It is obvious that both peak and mean BOD concentrations from the storage/treatment system are reduced sharply by arbitrarily increasing the magnitude of the longitudinal dispersion coefficient. For example, when E is increased 50-fold, from 72 ft^2/hr to 3600 ft^2/hr (6.7 to 334.8 m^2/hr), the peak BOD concentration is reduced by 76 percent and the mean BOD concentration by 57 percent.

When a reactor system is characterized by non-ideal mixing conditions, such as prevail in the dispersive storage/treatment case, it is convenient to define a dimensionless parameter (Wehner and Wilhelm, 1956).

$$Pe = \frac{L^2}{t_D E} \qquad (5.66)$$

where Pe = Peclet dispersion number.

The Peclet number is the inverse of the dimensionless dispersion number adopted by Thirumurthi (1969). An ideal reactor with fluid flowing through a long rectangular tank (or pipe, freshwater stream) with no longitudinal dispersion or lateral diffusion is characterized by an infinite dispersion number ($Pe = \infty$). At the other extreme is the limiting case of a fully stirred reactor

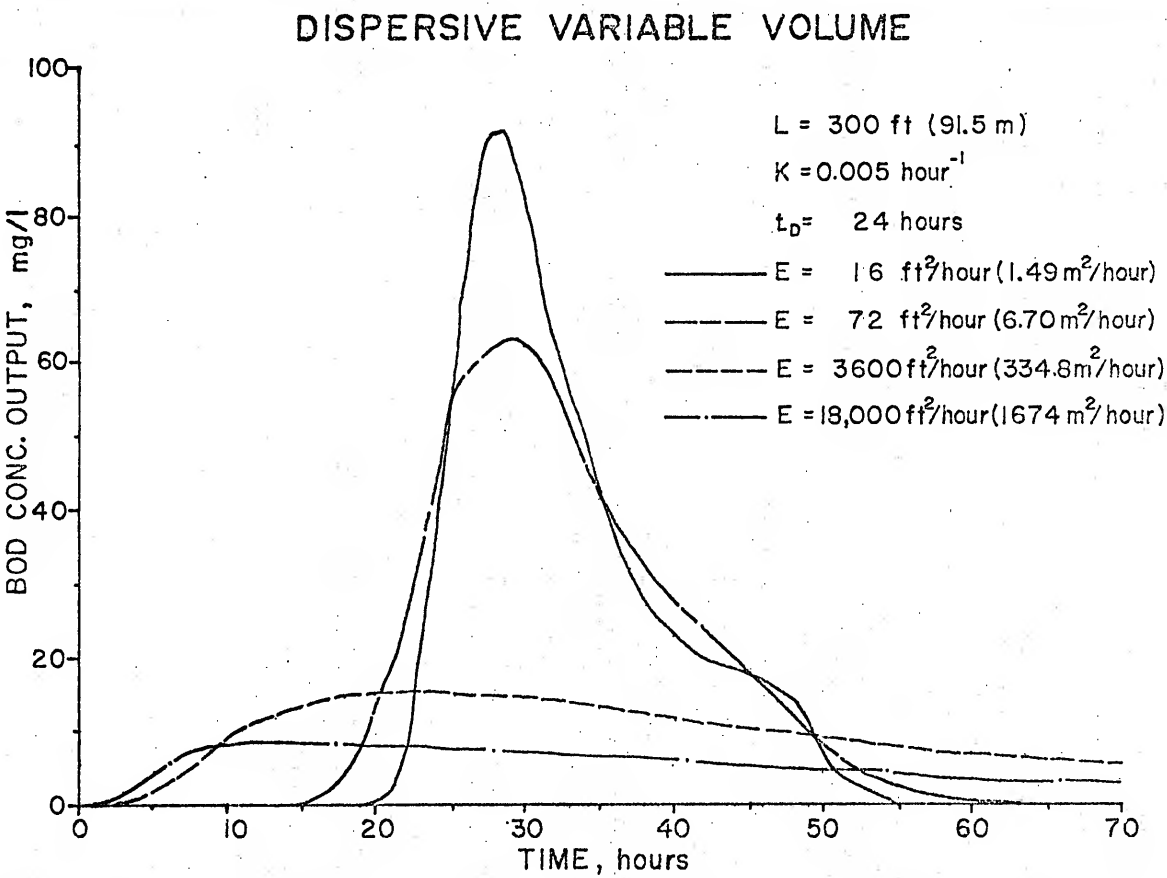


Figure 5-33. System BOD Concentration Response for Varied Longitudinal Dispersion Coefficient.

(activated sludge aeration tank, completely mixed detention pond, etc.) with infinite dispersion and a zero Peclet number. The dispersive storage/treatment system is analyzed at two intermediate stages, Pe = 52 and Pe = 104, by varying the three parameters that define the Peclet number. Figure 5-34 illustrates the system response to BOD concentration inputs from Wet Weather Event No. 52 for

(1) Pe = 104 where L = 1200 ft (366 m), t_D = 6 hours, and E = 2304 ft^2/hr (214 m^2/hr);

(2) Pe = 104 where L = 3600 ft (1097 m), t_D = 24 hours, and E = 5184 ft^2/hr (481 m^2/hr); and

(3) Pe = 52, t_D = 24 hours where

  (i) L = 300 ft (91.5 m), E = 72 ft^2/hr (60 m^2/hr); and

  (ii) L = 900 ft (274 m), E = 648 ft^2/hr (60 m^2/hr); and

  (iii) L = 3600 ft (1097 m), E = 10,368 ft^2/hr (963 m^2/hr).

It is interesting to note that systems with identical Peclet numbers but subjected to different detention times exhibit widely varying responses. Systems with different Peclet numbers but subjected to identical detention times exhibit responses which are much closer together. Finally,

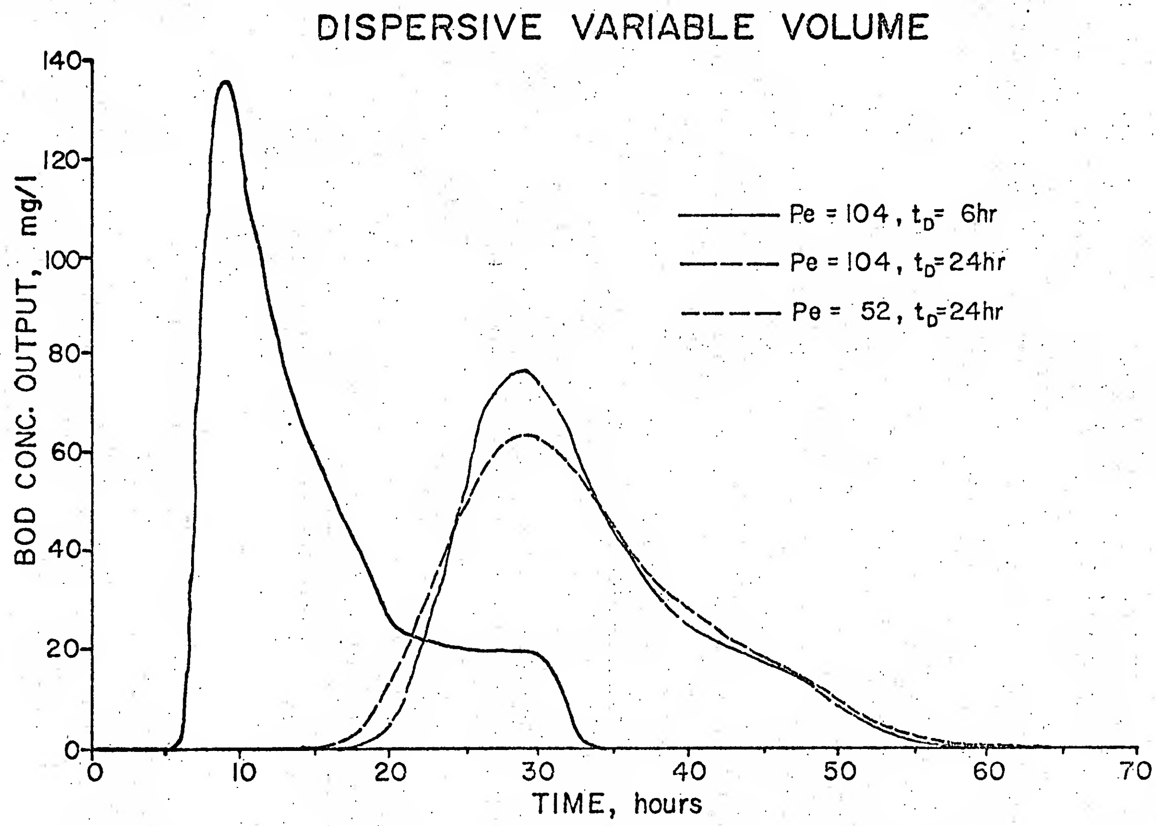


Figure 5-34. BOD Concentration Response for a Reactor Characterized by Peclet Numbers.

systems with identical Peclet numbers and subjected to identical detention times exhibit identical responses.

The above analysis may be extended by examining input and output statistics for BOD concentration for all wet weather events, presented in Table 5-21. The same conclusions may be drawn, so it appears that generalizations are possible among systems of different physical dimensions and dispersive characteristics. It is also interesting to examine the system response in terms of another dimensionless parameter, the reaction rate group $K\ t_D$. As stated previously, biological waste treatment systems are designed to be highly reactive relative to the detention time, and thus $K\ t_D >> 1$. The results of varying both parameters that define the rate group are presented in Table 5-22, for the storage/treatment system, for all wet weather events. In all cases $K\ t_D$ is much smaller than unity, contrary to secondary treatment biological waste systems. However, the basin mixing effects also contribute to reductions in peak and mean BOD concentrations. Unless very high reaction rate coefficients, K, are assumed for the pollutant mass it appears that the detention time provided by the dispersive storage/treatment system is the dominant factor in effluent quality.

Table 5-21

BOD Concentration Statistics for All
Wet Weather Events for Various Peclet Numbers

Item	Input to System	Response, % Reduction		
		Pe=104, t_D=6 hrs	Pe=104, t_D=24 hrs	Pe=52, t_D=24 hrs
Peak BOD Concentration	352 mg/l	209, 41%	130, 63%	106, 70%
Mean BOD Concentration*	64 mg/l	27, 58%	10, 84%	8, 87%

*Flow-weighted average.

Table 5-22

BOD Concentration Statistics for All Wet Weather Events for Various Reaction Rate Groups*

Item	Input to System	Response; % Reduction from Input				
		$Kt_D=0.06$	$Kt_D=0.06$	$Kt_D=0.12$	$Kt_D=0.12$	$Kt_D=0.12$
		$K=0.01\ hr^{-1}$ $t_D=6$ hrs	$K=0.005\ hr^{-1}$ $t_D=12$ hrs	$K=0.02\ hr^{-1}$ $t_D=6$ hrs	$K=0.01\ hr^{-1}$ $t_D=12$ hrs	$K=0.005\ hr^{-1}$ $t_D=24$ hrs
Peak BOD Concentration	352 mg/l	227; 36%	177; 50%	214; 39%	167; 53%	106; 70%
Mean BOD Concentration**	64 mg/l	28; 56%	18; 72%	26; 59%	17; 73%	8; 87%

*For a dispersive storage/treatment system characterized by L = 300 ft (91.5 m) and E = 72 ft^2/hr (6.7 m^2/hr).

**Flow-weighted average.

5.5 Summary

A rigorous mathematical approach has been presented to study the response of three storage/treatment systems to variable forcing functions. All of the models are based on the principle of conservation of mass. Thus, each system model is represented by a special case of the continuity equation, and its solution constitutes the system response. Nevertheless, intrinsic differences exist among the model parameters. For example, the well-mixed systems are characterized by a volume which is constant in one model and variable in the other. The variable volume models are characterized by completely mixed conditions in one and nonideal mixing in the other. In particular, the dispersive variable volume model requires a length dimension and a longitudinal dispersion coefficient. Therefore, the response of the various system models to a given set of inputs may vary considerably.

To illustrate these differences, system responses which were originally presented individually are now plotted together in Figures 5-35 and 5-36 for one detention time. The BOD inputs are from the single storm event. For the well-mixed cases, an immediate response is obtained at the system outfall because the reactor contents are assumed to be always completely mixed.

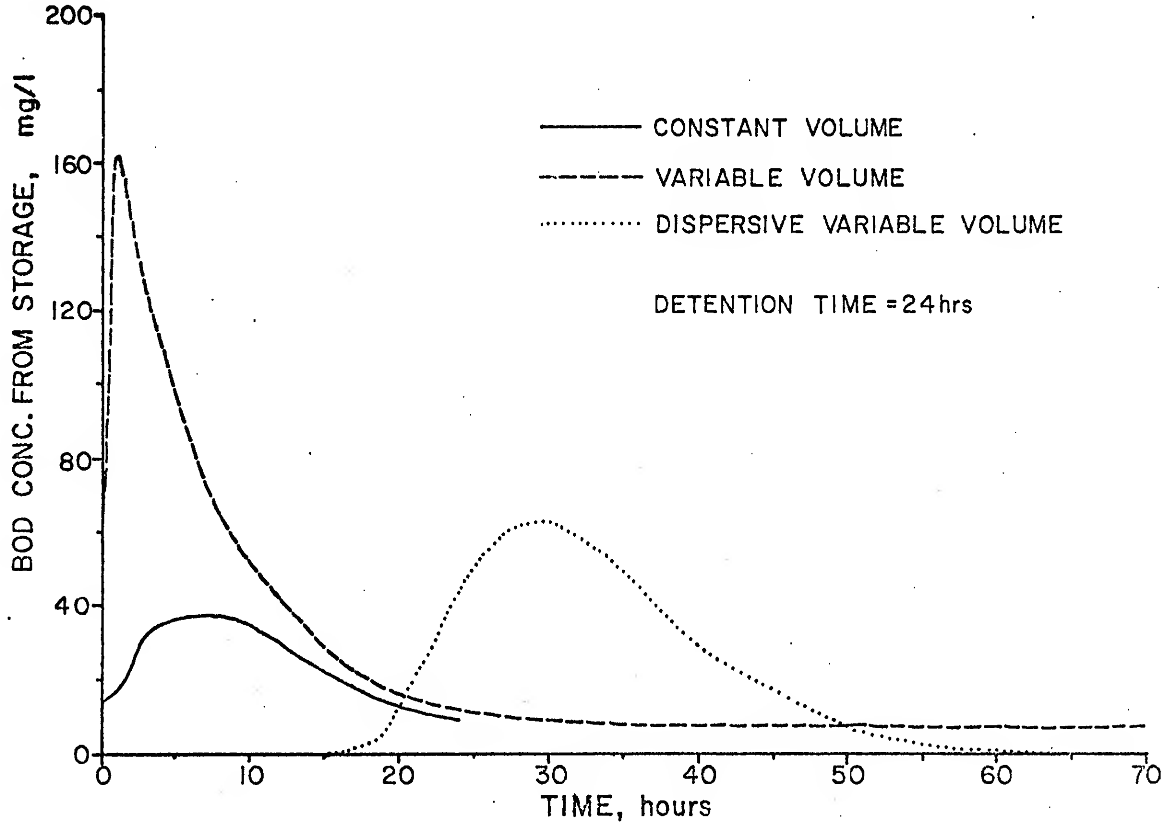


Figure 5-35. BOD Concentration Response for Various System Models.

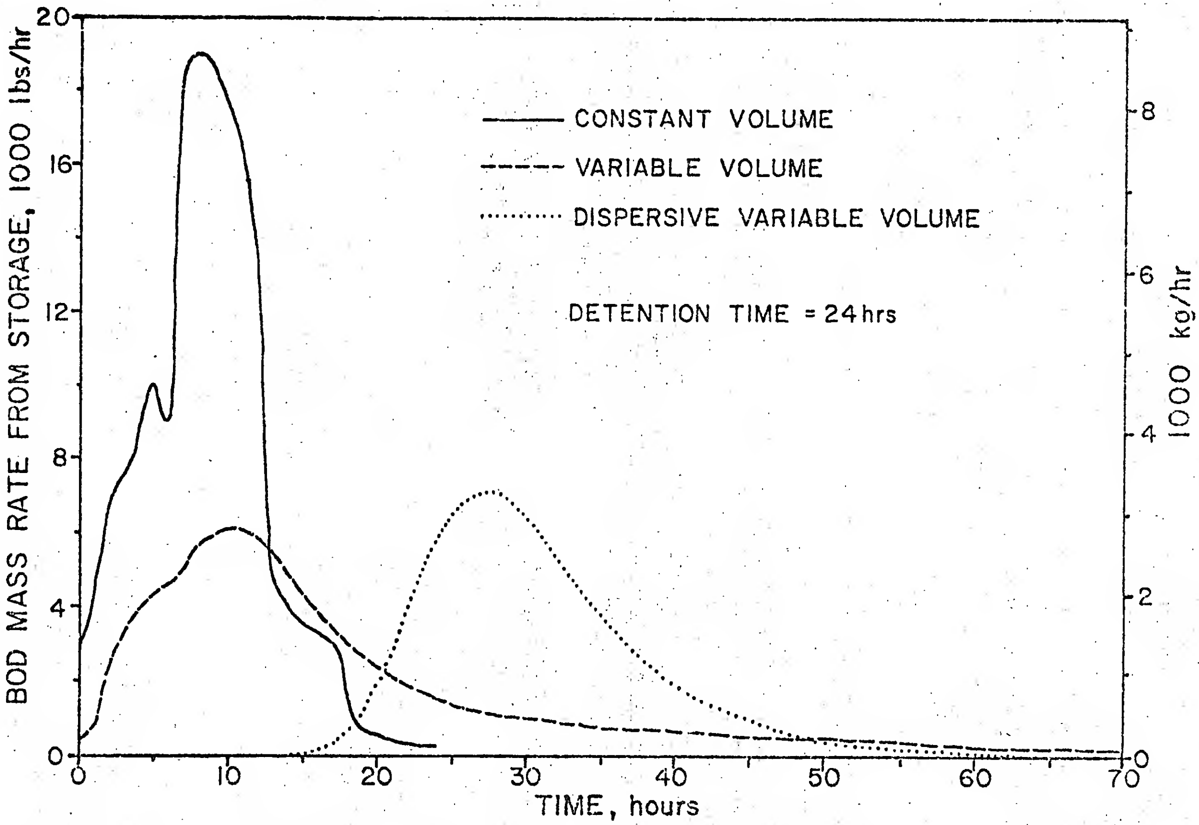


Figure 5-36. BOD Mass Rate Response for Various System Models.

For the dispersive case, the displacement of the response curve on the time scale is a function of the system longitudinal velocity and the longitudinal dispersion coefficient. Thus, a response at the system outfall depends on the length of the reactor, the detention time, and the magnitude of longitudinal dispersion in the system. For example, in Figures 5-35 and 5-36 the effects of the first pollutant injection are noticeable at the outfall approximately 15 hours later.

The BOD concentration curves in Figure 5-35 represent actual effluent concentrations. The constant volume storage/treatment system is more efficient at reducing peak and mean BOD concentrations than the other systems. In any well-mixed constant volume basin providing sufficiently long detention for inflows, concentration equalization is accomplished by both volumetric mixing and decay. The primary function of variable volume systems is to provide flow equalization. However, by reducing peak and mean outflow rates the pollutant peak and mean mass rates from these systems are effectively decreased as shown in Figure 5-36. The same pattern holds for the other residence times investigated, except that the performance of all three system models improves steadily as the detention time is increased. The high peak in the well-mixed variable volume response curve (Figure 5-35) is due to initial conditions in the

basin. These conditions cannot be accounted for by the dispersive variable volume model, and therefore the response for a high longitudinal dispersion coefficient (see Figure 5-33) does not approximate that of the well-mixed variable volume system.

Unquestionably, whether the systems are well-mixed or dispersive, the dominant factor in pollutant removal efficiency is the detention time. This means that, for either constant volume or variable volume models, the detention facility size is of major significance. Based on modeling studies using performance data obtained from the Humboldt Avenue combined sewer overflow detention tank, the City of Milwaukee et al. (1975) predicted BOD removal efficiencies of about 35 percent for a tank volume of 1 million gallons/sq. mi of drainage area (1461 cu m/sq km). The BOD removal efficiency increased to about 85 percent for a tank volume of 6 million gallons/sq. mi (8766 cu m/sq km). These results were projected for an annual precipitation total of 28.9 inches (734 mm). Since the facility simply overflowed to the receiving water when inflows exceeded tank capacity, it may be compared to the well-mixed constant volume storage/treatment model applied to Des Moines, Iowa, for which an annual precipitation total of 27.59 inches (701 mm) was recorded.

For an average detention time of 6 hours, the constant volume model achieved a mean BOD concentration removal efficiency of 42 percent (see Table 5-7) for a tank volume of 1.26 million gallons per square mile of drainage area (1842 cu m/sq km) for all wet weather events during the year. The BOD removal efficiency increased to 70 percent for a tank volume of 5.1 million gallons per square mile (7372 cu m/sq km) which maintained an average detention time of 24 hours. The close agreement between results obtained for the Milwaukee, Wisconsin facility and the model application to Des Moines, Iowa has great significance. It provides a certain degree of confidence in generalizing the findings obtained throughout this chapter.

Considering the importance of preventing shock BOD loadings to receiving waters as well as providing adequate reductions in mean BOD concentrations, it appears that 6 to 12 hours of detention or higher should be given to urban runoff flows prior to discharge. The requirements will vary according to the BOD strength of the urban stormwater surface runoff, mixing in combined sewers, and receiving water conditions. The effects on receiving water minimum dissolved oxygen levels produced by varying the detention time of a constant volume storage/treatment system are evaluated for the Des Moines River in Chapter VII.

CHAPTER VI

STATISTICAL APPROACH TO STORAGE/TREATMENT RESPONSE FOR WELL-MIXED CONSTANT VOLUME MODEL

6.1 Introduction

A deterministic approach to the transient response of a well-mixed constant volume storage/treatment system, based on the unifying principle of continuity, has been presented in Chapter V. The statistical approach addressed herein is complementary to the mathematical solutions to equation (2.8), and applies only to a system governed by equation (2.8). The methodology discussed in this chapter provides, specifically, a computationally convenient measure of equalization of time variable waste loads. When the statistical approach is not used in a complementary role, some serious limitations may arise depending on the overall objectives.

6.2 Equalization of Concentration and Mass Flow Rate

Equalization may be defined as a means of dampening extreme fluctuations of concentration and mass flow rate in wastewater, by providing a storage/treatment facility

with a sufficiently long detention time for complete volumetric mixing and adequate decay. Numerous investigators have addressed the need to provide equalization facilities for wastewater treatment plants. Effluents from industrial processes often contain components whose concentrations are highly variable in time (Wallace, 1968). Municipal sewage discharges are also variable phenomenons: extreme low flows occur between 2 and 6 A.M., and the maximum flow occurs at about noon (Metcalf & Eddy, Inc., 1972). Engineering experience has shown that biological treatment processes, and even chemical treatment trains, perform better if extreme fluctuations in the load to the reactor system can be avoided or damped (Eckenfelder, 1966). Failure to account for the unsteady nature of influent waste loads can cause severe deterioration of the treatment efficiency and impairment of receiving water quality (Novotny and Stein, 1976).

An examination of the hydrologic time series depicted in Figure 4-1 is sufficient to confirm that extreme fluctuations are found in urban stormwater runoff. The WWF storage/treatment system (refer to Figure 3-4) is subjected to variability in the influent mass rate and also fluctuations in the fluid flow rate. Wallace (1968) and later Novotny and Stein (1976) proposed a wide variety of analytical procedures to characterize output variance.

These techniques revolve around the definition of an equalization function which is the ratio of the variance of the output concentration-time curve to the variance of the input concentration-time curve. Novotny and Stein, in particular, use power spectral analysis to describe equalization of concentration for completely mixed basins, dispersed flow basins, and plug flow basins. However, both Wallace, and Novotny and Stein, assume a constant fluid flow rate through the reactor. Thus, such techniques are unsuitable for adaptation to urban stormwater runoff storage/treatment systems because they are not applicable for the case of varying flow.

In a recent paper by DiToro (1975), the author presents an analytic framework that addresses the design of a completely mixed constant volume equalization basin receiving a fluctuating influent mass flow rate and a fluctuating influent fluid flow rate. Such a framework is adopted in this chapter to investigate the degree of equalization provided by WWF storage/treatment systems for the well-mixed constant volume case.

6.3 Methodology

It is quite difficult to determine the actual probability density function (pdf) of the time-varying input concentration, $C_1(t)$, even for simple probabilistic

models. The analysis is complicated even further if consideration must be given to essential hydrologic variabilities. These variabilities consist of (1) heterogeneities of catchment properties, (2) heterogeneities of individual storm properties, and (3) random variabilities in the climatic time series of storms and antecedent moisture conditions (Eagleson, 1972). The only case for which the pdf is known is for <u>constant</u> flow rate for which it can be shown that a Gaussian mass rate input results in an effluent concentration which is also Gaussian (DiToro, 1975). Thus the pdf of the input mass flow rate, $W_1(t)$, is ignored. Instead, the inputs are described by basic statistical averages and exponentially decaying serial correlation coefficients.

The relationships that follow apply to a well-mixed constant volume storage/treatment system, governed by equation (2.8) or (5.6), expressed as

$$V \frac{dC_2}{dt} = W_1(t) - Q(t)C_2(t) - K\, C_2(t)\, V \qquad (6.1)$$

where $W_1(t)$ = fluctuating influent mass rate, M/T,

$Q(t)$ = fluctuating influent fluid flow rate, L^3/T,

$C_2(t)$ = time-varying effluent concentration, M/L^3,

K = first-order decay coefficient, 1/T, and

V = time invariant volume of the fluid mass in the system, L^3.

With the introduction of probabilistic inputs and coefficients the problem becomes one of analyzing random differential equations, but the exact solutions for such problems are seldom available or quite complex so that an approximate analysis is all that is possible. The basic statistical measures have already been presented in Chaper IV. However, the notation was of a general nature, and it is convenient to redefine the variables again as follows:

$$\overline{W}_1 = \frac{1}{n} \sum_{i=1}^{n} W_1(t_i) \tag{6.2}$$

$$\overline{Q} = \frac{1}{n} \sum_{i=1}^{n} Q(t_i) \tag{6.3}$$

$$\sigma_W^{\ 2} = \frac{1}{n-1} \sum_{i=n}^{n} \left[W_1(t_i) - \overline{W}_1 \right]^2 \tag{6.4}$$

$$\sigma_Q^{\ 2} = \frac{1}{n-1} \sum_{i=1}^{n} \left[Q(t_i) - \overline{Q} \right]^2 \tag{6.5}$$

$$\nu_W = \frac{\sigma_W}{\overline{W}_1} \tag{6.6}$$

$$\nu_Q = \frac{\sigma_Q}{\overline{Q}} \tag{6.7}$$

where $\overline{W}_1$ = mean influent mass rate, M/T,

$\overline{Q}$ = mean influent fluid flow rate, L^3/T,

σ_W^2 = variance of the influent mass rate, M^2/T^2,

σ_Q^2 = variance of the influent fluid flow rate, L^6/T^2,

σ_W = standard deviation of the influent mass rate, M/T,

σ_Q = standard deviation of the influent fluid flow rate, L^3/T,

ν_W = coefficient of variation of the influent mass rate, dimensionless, and

ν_Q = coefficient of variation of the influent fluid flow rate, dimensionless.

Although the lag k serial correlation for the influent mass flow rate, $r_W(k)$, may be defined as (DiToro, 1975)

$$r_W(k) = \frac{1}{n-k} \sum_{i=1}^{n-k} \left[\frac{W_1(t_i) - \overline{W}_1}{\sigma_W}\right] \left[\frac{W_1(t_{i+k}) - \overline{W}_1}{\sigma_W}\right] \tag{6.8}$$

actual computation herein is performed by using equation (4.1) as recommended by Yevjevich (1972) and Fiering and

Jackson (1971). Assuming exponentially decaying serial correlation coefficients (DiToro, 1975),

$$r_W(k) = \exp\left[-\frac{kh}{T_W}\right] \tag{6.9}$$

where k = hourly lag, dimensionless,

h = sampling interval

$= t_{i+1} - t_i = 1$ hour, and

T_W = correlation time constant, for mass rate inputs, hours.

The correlation time constant T_W is essentially the time interval that separates uncorrelated random influent mass rates, $W_1(t)$ and $W_1(t+T_W)$. The value of T_W is obtained by fitting equation (6.9) to the correlogram obtained from computation of equation (6.8). This procedure is demonstrated in a later application. The lag k serial correlation coefficient for the fluid flow rate, $r_Q(k)$, is similarly defined by equations (6.8) and (6.9) using the appropriate substitutions. Thus,

$$r_Q(k) = \exp\left[-\frac{kh}{T_Q}\right] \tag{6.10}$$

where T_Q = correlation time constant for the influent fluid flow rates, hours.

DiToro (1975) states that an influent fluid flow rate which is fluctuating in a random way increases both the mean effluent concentration and the effluent concentration variation over that expected with no flow variation. It is reasonable to expect an increase in the effluent concentration variance. The explanation suggested for the increase in the mean effluent concentration is that the average value of the ratio of two random variables, $W_1(t)/Q(t)$, is not equal to the ratio of the averages. However, DiToro assumed that $Q(t)$ and $W_1(t)$ are statistically independent. The author of this work demonstrates in a subsequent application that these variables may be significantly correlated when dealing with urban stormwater runoff. Normalized detention times are defined as

$$\tau_W = \frac{\bar{t}_D}{T_W (1 + K \bar{t}_D)} \tag{6.11}$$

$$\tau_Q = \frac{\bar{t}_D}{T_Q (1 + K \bar{t}_D)} \tag{6.12}$$

where τ_W = normalized detention time for the pollutant mass input, dimensionless, and

τ_Q = normalized detention time for the fluid flow, also dimensionless.

The appearance of the term $(1 + K\,\bar{t}_D)$ in the denominator of equations (6.11) and (6.12) is from a well-known result of the solution to the conservation of mass equation, shown in equation (2.9).

From an approximate analysis of equation (6.1) with random inputs $W_1(t)$ and $Q(t)$, and assumption of the exponential correlation given by equations (6.9) and (6.10), DiToro defines a flow variation parameter

$$m = \frac{\nu_Q}{(1 + K\,\bar{t}_D)\sqrt{1 + \tau_Q}} \tag{6.13}$$

where m = equalized variation of the flow, dimensionless.

Finally, the effluent mean concentration and the effluent concentration coefficient of variation may be defined in terms of the flow variation parameter, m:

$$\bar{C}_2 = \frac{\bar{W}_1}{\bar{Q}\,(1 + K\,\bar{t}_D)\,(1-m^2)} \quad , \; m < 1 \tag{6.14}$$

and

$$\nu_C = \left[\frac{\nu_W^2\,(1-m^2)^2}{(1 + \tau_W)} + m^2\right]^{0.5} \quad , \; m < 1 \tag{6.15}$$

where $\overline{C}_2$ = mean effluent concentration, M/L^3, and

ν_C = coefficient of variation of the effluent concentration, dimensionless.

It is interesting to note that equation (6.14) is identical to the steady-state solution to equation (2.8), for $m = 0$. In other words, m is merely a correction factor to account for the decreased equalization in the basin due to fluid flow rate fluctuations. It should be noted that equations (6.14) and (6.15) are approximate in that they apply for $m < 1$ with the approximation deteriorating as $m \rightarrow 1$ (DiToro, 1975). If it can be assumed that the effluent concentration, $C_2(t)$, and the fluid flow rate, $Q(t)$, are jointly Gaussian, the coefficient of variation of the effluent mass flow rate may be computed from

$$\nu_{CQ} = \frac{\nu_C \, \nu_Q}{1 - m^2} \sqrt{1 + \frac{1}{\nu_C^2} + \frac{1}{\nu_Q^2} + \frac{m^2(m^2-2)}{\nu_C^2 \, \nu_Q^2}} \qquad (6.16)$$

where ν_{CQ} = coefficient of variation of the effluent mass flow rate, dimensionless.

In the derivation of equations (6.13) through (6.15), it has been assumed that $Q(t)$ and $W_1(t)$ are statistically independent. The cross-correlation of these two variables, at lag zero, is the ordinary (product-moment) correlation coefficient and may be estimated by (Yevjevich, 1972)

$$r_{WQ} = \frac{\frac{1}{n}\sum_{i=1}^{n} W_1(t_i)Q(t_i) - \frac{1}{n^2}\sum_{i=1}^{n} W_1(t_i)\sum_{i=1}^{n} Q(t_i)}{\left[\frac{1}{n}\sum_{i=1}^{n} W_1^{\,2}(t_i) - \frac{1}{n^2}\left[\sum_{i=1}^{n} W_1(t_i)\right]^2\right]^{0.5} \cdot \left[\frac{1}{n}\sum_{i=1}^{n} Q^2(t_i) - \frac{1}{n^2}\left[\sum_{i=1}^{n} Q(t_i)\right]^2\right]^{0.5}} \qquad (6.17)$$

Equation (6.17) should be used to test the independence of the probabilistic inputs.

6.4 Application to Des Moines, Iowa

The statistical approach discussed in the previous section is applied to the series of urban runoff inputs generated by precipitation over Des Moines, Iowa (refer to Figure 4-1). The response of the well-mixed constant volume storage/treatment system to these inputs is characterized primarily by the mean BOD concentration of the effluent and its coefficient of variation. The influent BOD mass rate and fluid flow rate are characterized by temporal averages presented in Table 6-1. The correlation time constants, T_W and T_Q, are obtained by fitting exponential curves to the correlograms obtained from computation of serial correlation coefficients.

Table 6-1

Summary Statistics for BOD Mass Rate and Fluid Flow Rate Inputs of the Urban Runoff Time Series*

Statistics	Computed Value
Mean Flow Rate, $\overline{Q}$	1,584 cfs (45 cu m/sec)
Standard Deviation of the Flow Rate, σ_Q	1,290 cfs (37 cu m/sec)
Coefficient of Variation of the Flow Rate, ν_Q	0.81
Mean BOD Mass Rate, $\overline{W}_1$	21,717 lbs/hr (9,851 kg/hr)
Standard Deviation of BOD Mass Rate, σ_W	18,250 lbs/hr) (8,278 kg/hr)
Coefficient of Variation of BOD Mass Rate, ν_W	0.84
Correlation Time Constant for Mass Rate, T_W	1.10 hours
Correlation Time Constant for Fluid Flow Rate, T_Q	1.40 hours

*Precipitation year of record for Des Moines, Iowa, is 1968.

Figure 6-1 shows the correlogram for the input BOD mass rate, for all wet weather events. Tolerance limits at the 95 percent probability level (5 percent probability of committing a Type I error) are indicated. As stated in Chapter IV, values of the function between these limits are essentially uncorrelated. The exponential form of the serial correlation coefficient function is approximately

$$r_W(k) = \exp\left[-\,0.91\ k\right] \tag{6.18}$$

where T_W is found to be 1.10 hours. Similarly, from the correlogram for the flow rate, shown in Figure 6-2, its exponential form is approximately given by

$$r_Q(k) = \exp\left[-\,0.71\ k\right] \tag{6.19}$$

where T_Q is found to be 1.40 hours.

Equations (6.13) through (6.16) may be applied, from the information given in Table 6-1, to the storage/treatment system for the residence times of interest. The first-order decay constant, K, is determined from equations (5.19), (5.21), and (5.23). The statistical characterization of the storage/treatment effluent, and other pertinent parameters, are summarized in Table 6-2. In particular, the response of an equalization basin to waste inputs is

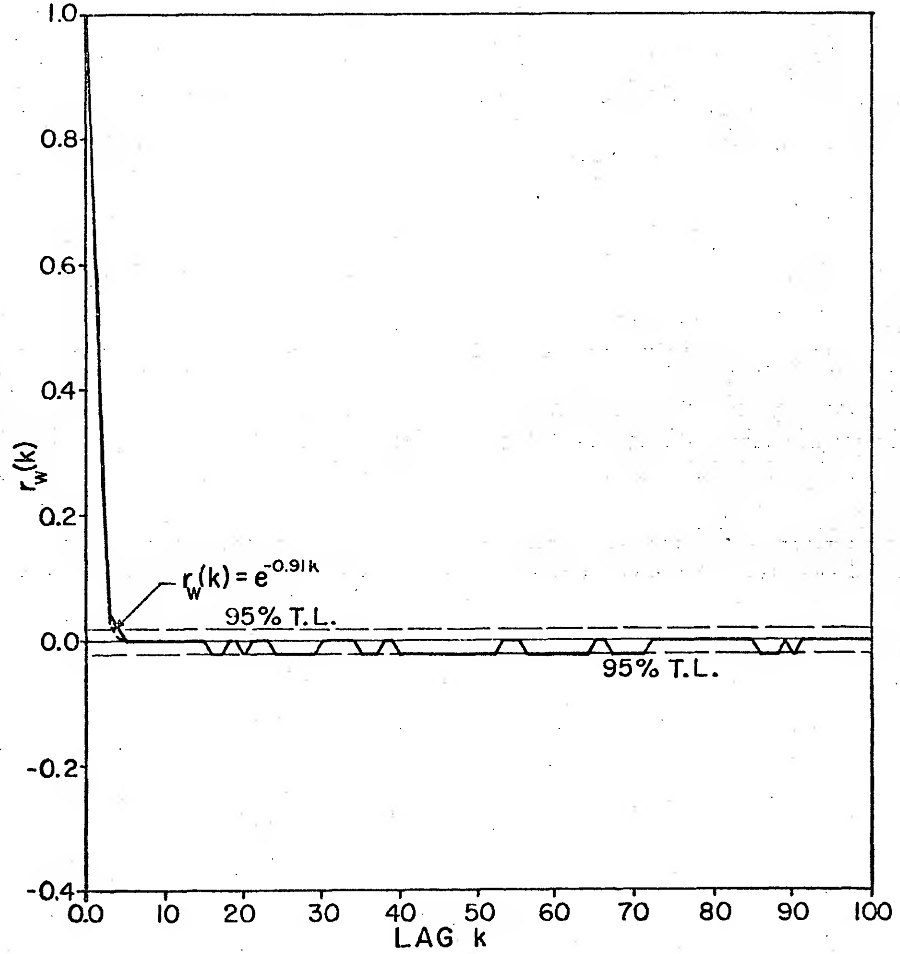


Figure 6-1. Lag k Serial Correlation Coefficients for Input BOD Mass Rates.

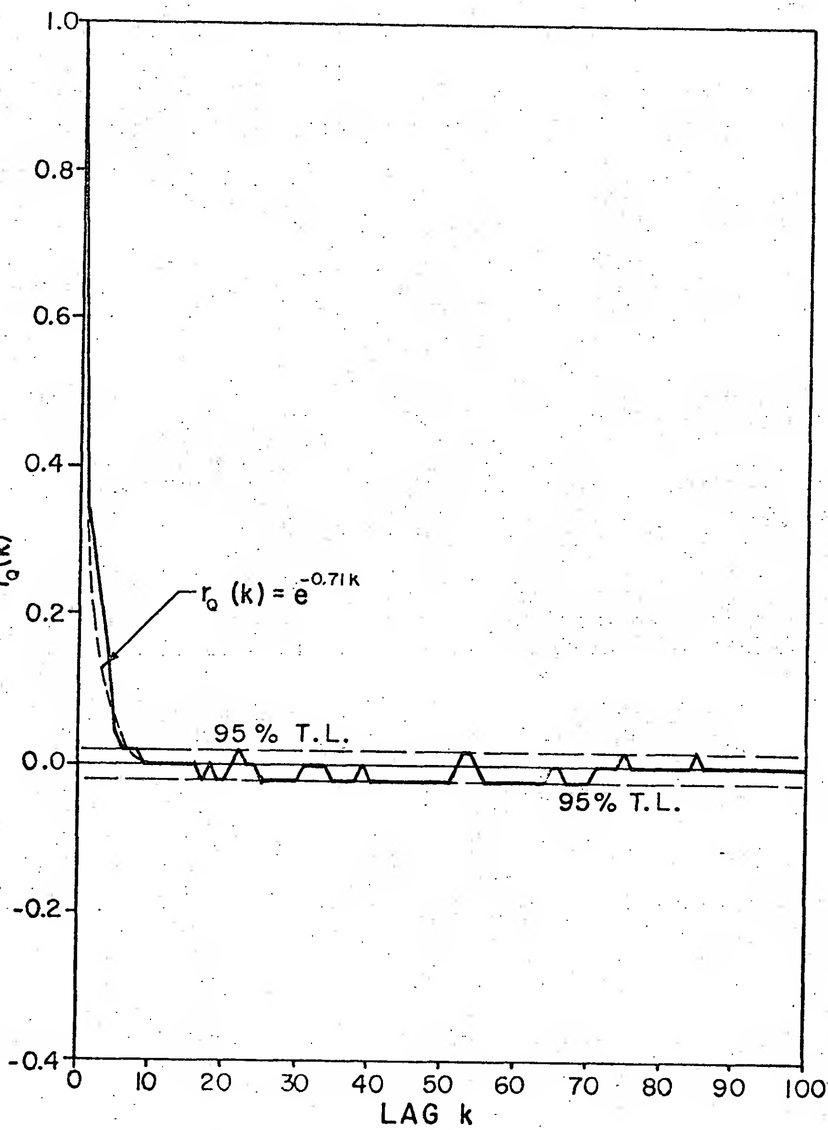


Figure 6-2. Lag k Serial Correlation Coefficients for Input Fluid Flow Rates.

Table 6-2

Predicted System Response to Mass and Fluid Flow Rate Inputs

Parameter	Storage Treatment Response			
	$\bar{t}_D$=2 hrs	$\bar{t}_D$=6 hrs	$\bar{t}_D$=12 hrs	$\bar{t}_D$= 24 hrs
First-Order Decay Rate Constant,* K	0.083 hours^{-1}	0.109 hours^{-1}	0.110 hours^{-1}	0.110 hours^{-1}
Dimensionless Fluid Detention Time, τ_Q	1.23	2.59	3.69	4.70
Dimensionless Mass Detention Time, τ_W	1.56	3.30	4.70	5.98
Equalized Variation of Flow, m	0.47	0.26	0.16	0.09
Mean Effluent BOD Concentration, $\bar{C}_2$	67 mg/l	39 mg/l	27 mg/l	17 mg/l
Coefficient of Variation of Effluent Concentration, ν_C	0.62	0.46	0.39	0.33
Coefficient of Variation of Effluent Mass Rate, ν_{CQ}**	1.22	1.00	0.95	0.91

*Increases with detention time due to increased sedimentation of suspended solids.

**Assumes that effluent concentration and fluid flow rate are jointly Gaussian.

important to receiving water quality in two aspects: reduction of both mean concentrations and concentration variabilities. The predicted pollutant removal efficiencies of the well-mixed constant volume model are presented in Table 6-3. A relative degree of equalization may be estimated from the percentage reduction between the coefficient of variation of the influent BOD mass rate, ν_W, and the coefficient of variation of the effluent BOD concentration, ν_C (Novotny and Stein, 1976).

From Table 6-3, it appears that the approximation provided by equation (6.14) deteriorates for a value of m much smaller than unity when the average detention time $\bar{t}_D$ is small. By inspection of equation (6.13), it can be seen that m is sensitive to the magnitudes of the average detention time and the normalized detention time for fluid flow. The statistical model predicts an increase in the mean BOD concentration due to a fluctuating fluid flow rate, which is accounted for in equation (6.14) by the term $(1 - m^2)$. A constant flow rate would result in an effluent mean BOD concentration of 52 mg/l. However, the statistical model also predicts a coefficient of variation of the effluent BOD concentration of 0.62 versus 0.84 for the influent BOD mass rate. It should be emphasized that the numerical difference between ν_C and ν_W is not an absolute measure of the degree of equalization. Otherwise,

Table 6-3

Pollutant Removal Efficiency of the Well-Mixed Constant Volume Model

Average Detention Time	% Reduction in Mean BOD Concentration*	Equalization $\left[\frac{\nu_W - \nu_C}{\nu_W}\right] 100, \%$
$\bar{t}_D$ = 2 hours	-10	26
$\bar{t}_D$ = 6 hours	36	45
$\bar{t}_D$ = 12 hours	55	54
$\bar{t}_D$ = 24 hours	72	61

*The statistical average of the influent BOD concentration is computed by the ratio $\overline{W}_1/\overline{Q}$ and is equal to 61 mg/l.

agreement between the mean effluent BOD concentrations as predicted by the deterministic model (Table 5-7) and the statistical approach (Table 6-2) is quite good. To illustrate, these values are plotted, over the same range of detention times, in Figure 6-3.

Close examination of the other items in Table 6-2 provides more insight into the behavior of the storage/treatment system. If the detention time of the basin ($\bar{t}_D$) is much larger than the correlation time of the fluid flow rates (T_Q), then the dimensionless detention time τ_Q is large and the flow fluctuations are smoothed by the basin mixing effect. For example, the flow variation parameter, m, decreases as τ_Q increases according to the relationship in equation (6.13). Thus, for an average detention time of 24 hours and the urban runoff time history for Des Moines, Iowa, for 1968, (1) a value of 0.99 is obtained for the term $(1 - m^2)$, so that (2) the flow variation has little effect on the computation of the mean effluent concentration in equation (6.14). This is not surprising at all, since it is merely a mathematical description of the fact that a large enough storage/treatment facility will attenuate the extreme fluctuations in the influent fluid flow rates, eliminating interference with the simultaneous process of mixing and first-order decay of the influent pollutant mass rates. Of course, the inverse situation occurs when T_Q increases relative to $\bar{t}_D$: flow fluctuations

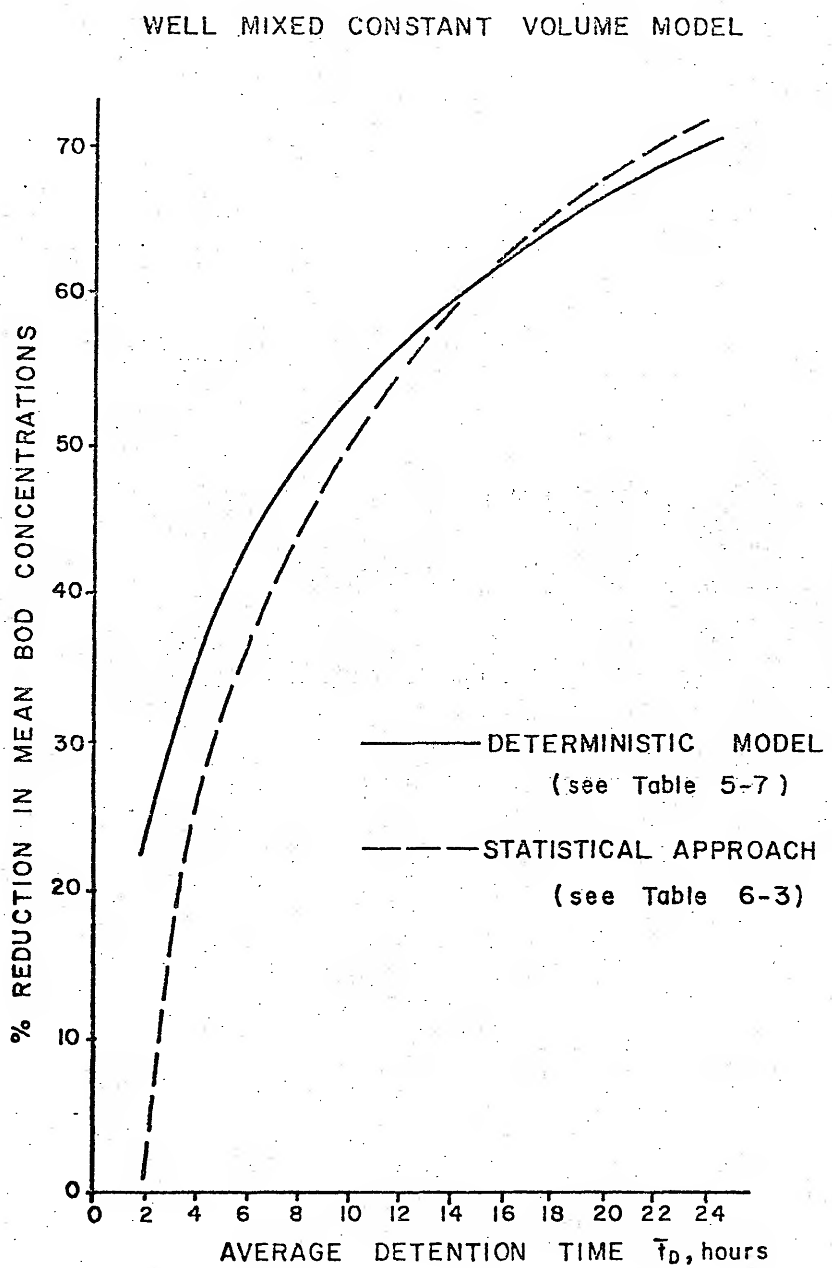


Figure 6-3. Deterministic Model and Statistical Approach Predictions for All Wet Weather Events.

become a significant source of variation, increasing both the mean concentration and the variation of the effluent.

6.5 Limitations of a Strictly Statistical Approach

First of all, the statistical approach does not predict the peak concentrations which may be expected to enter the receiving water, nor the peak mass rates. This limitation may not be critical to the design of an equalization basin, but is very important to study the effects of shock loads on dissolved oxygen levels in the receiving body of water. Of course, the statistical approach does not provide a transient history of the response of the storage/treatment system either. Thus, it is most useful in the evaluation of equalization effects when the overall objectives are as stated in Chapter I.

The statistical approach is further limited when applied to urban stormwater studies in another respect: the influent mass rates and the fluid flow rates may be significantly correlated. Application of equation (6.17) resulted in a value for r_{WQ} of 0.79; consequently, the assumption of independence between $W_1(t)$ and $Q(t)$ is not supported. This fact does not appear to invalidate the analysis presented thus far in view of the comparison presented in Figure 6-3. However, it seems to indicate that probability statements concerning the effluent concentration

would be risky. Furthermore, application of the statistical approach to a single storm event proved impossible. Figure 6-4 shows a plot of the lag k serial correlation coefficients for the influent fluid flow rates of Wet Weather Event No. 52. Although the event has a runoff duration of 24 hours, an exponentially decaying curve cannot be fitted to the correlogram. The widely diverging tolerance limits indicate the region of essentially uncorrelated coefficients, not significantly different from zero. Thus, a much longer time history is required for analysis than provided by one event. Otherwise, a much smaller time interval than one hour is required. Observations at time intervals less than one hour are difficult to obtain in urban hydrologic studies (McPherson, 1974).

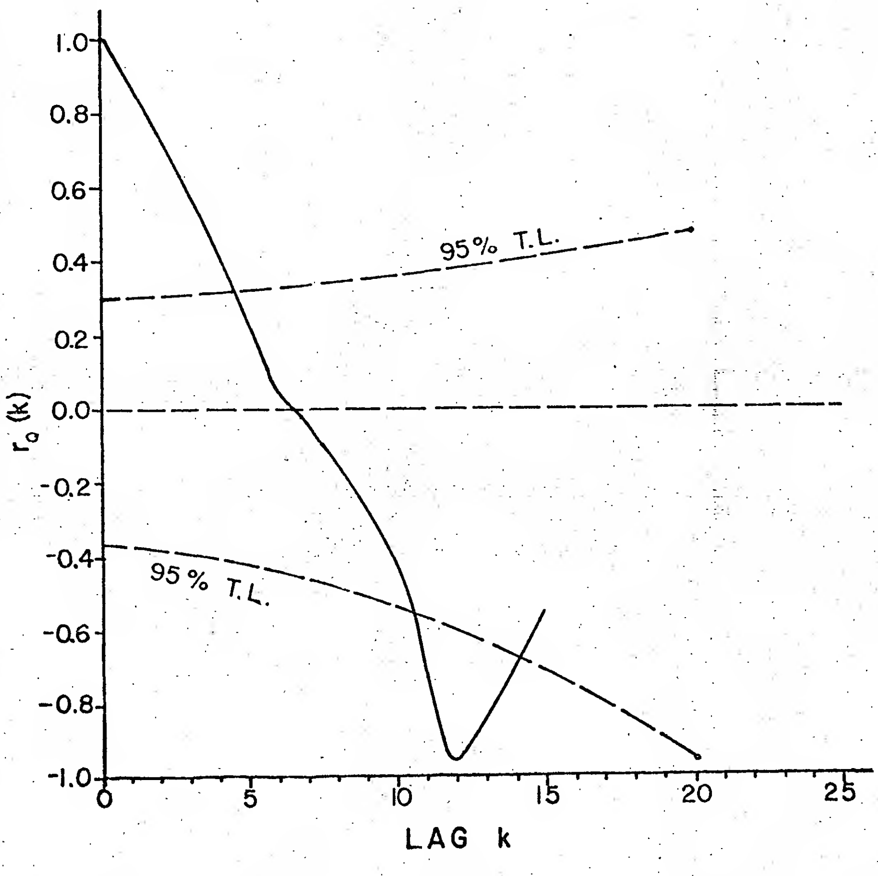


Figure 6-4. Lag k Serial Correlation Coefficients for Fluid Flow Rate for Wet Weather Event No. 52, Hourly.

CHAPTER VII

RECEIVING WATER RESPONSE TO WASTE INPUT COMBINATIONS

7.1 Introduction

The choice of receiving water analyses is wide-ranging, from simply examining wastewater concentrations at the inflow points to the use of detailed hydrodynamic and water quality simulation models. The methodology presented herein is consistent with the research objectives enumerated in Chapter I and the unified approach described in Chapter II. The receiving body of water may be viewed as a natural storage/treatment system and may be represented by the generalized component depicted in Figure 2-1. Simplified mathematical modeling techniques are used to determine critical deficits and resulting minimum dissolved oxygen (D.O.) concentrations for various waste input combinations and stream conditions, as summarized in Table 7-1. An abstraction on the physical system has been presented in Figure 3-4, which is useful to review in conjunction with Table 7-1.

The mathematical modeling techniques adopted throughout this chapter were applied by the author to

Table 7-1

Waste Input Combinations and Stream Conditions
(See Figure 3-4)

The combined effects of the following waste sources are modeled:

1. Upstream river flow

2. DWF treatment plant effluent

3. WWF storage/treatment facility effluent from separate sewer and combined sewer flow inputs.

For the following DWF treatment rates:

1. 30% BOD removal, primary treatment

2. 85% BOD removal, secondary treatment

For a well-mixed constant volume WWF storage/treatment facility maintaining the following average detention times:

1. t_D = 0, no storage/treatment

2. t_D = 2 hours

3. t_D = 6 hours

4. t_D = 12 hours

5. t_D = 24 hours

And the following stream conditions, measured upstream from the point of urban waste inflows are considered for each time step:

1. River flow

2. Initial BOD concentration

3. Initial D.O. concentration

4. River temperature

Des Moines, Iowa (Heaney et al., 1976) to simulate the hypothetical response of the Des Moines River to the separate and combined BOD waste inputs from (1) upstream sources, (2) dry weather urban sources, and (3) wet weather urban sources. Emphasis was placed on the relative importance of these waste inputs to receiving water minimum D.O. levels, as an example of a possible methodology for areawide wastewater management and planning. The presentation of results was of key importance, and continuous simulation provided a means by which cumulative D.O. frequency curves (on an annual basis) could be developed to demonstrate the deleterious impact of wet weather flows.

The emphasis placed herein is, as summarized in Table 7-1, to assess the adequacy of the well-mixed constant volume storage/treatment system as a pollutant control device during periods of urban runoff. The WWF storage/treatment system, the DWF treatment plant, and upstream river flow interact to produce a receiving water response below the urban area, as a function of stream conditions. The development of a detailed receiving water model is not justified for the problem context, since a much larger data base would have been necessary on river channel geometry and quantity and quality measurements.

Some key assumptions, most of them typical of models for interim planning, are made (Hydroscience, 1971).

1. Temporal steady-state conditions prevail, where all system parameters and inputs are constant with respect to time; however, a relatively short time step (1 hour) is used for simulation.
2. Stream system parameters (such as river flow, velocity, depth, deoxygenation rate, and reaeration rate) are spatially constant along the flow axis throughout each time step.
3. All waste inflows occur at one point on the receiving stream.
4. The effects of various natural biological processes (algal photosynthesis and respiration, benthal stabilization) are incorporated by D.O. deficit (if none, by saturation) upstream from the waste inflow point.
5. The DWF waste treatment plant operates at constant efficiencies, independent of hydraulic and organic loadings, for the entire period of simulation.

7.2 Initial Conditions

Initial conditions of BOD in the river are defined by equation (3.3). In subsequent equations, the mixed

BOD concentration in the river will be denoted by L_o. Thus,

$$L_o \equiv BOD_m \tag{7.1}$$

The assumption that all waste inflows occur at one point is not unreasonable for Des Moines, but in some locations the distribution of inflows along the river may need to be considered. It is important to emphasize that all of the BOD contributors in equation (2.4) represent BOD_5 values. Thus, the mixed BOD concentration in the river, BOD_m, is also in terms of the standard BOD test. The ultimate first-stage (carbonaceous) demand is related to the BOD_5 value by

$$(L_o)_c = \frac{BOD_5}{1-e^{-5K_1}} \tag{7.2}$$

where

$(L_o)_c$ = ultimate first-stage BOD demand, mg/l, and

K_1 = first-order BOD decay rate constant, day^{-1}.

Thus, the mixed BOD concentration in the river in terms of the ultimate BOD first-stage demand is given by

$$(L_o)_c = \frac{L_o}{1 - \exp\ (- 120\ K_1)} \tag{7.3}$$

where

K_1 = first-order BOD decay constant, hours^{-1}.

The other initial condition required is the initial oxygen deficit, D_0. It is assumed that all waste inflows will be at saturation. Thus, the only contribution to the initial deficit will be from the upstream portion of the river. Thus,

$$D_o = \frac{D_u Q_u}{Q_u + Q_d + Q_s + Q_c} \qquad (7.4)$$

where

D_o = initial D.O. deficit, mg/l, and

D_u = D.O. deficit in receiving waters upstream of inflow point, mg/l.

7.3 Oxygen Balance of Polluted Streams

Pollutant transport processes in a stream system may be adequately aproximated by the one-dimensional version of the classical convective dispersion equation. This partial differential equation is based on the principles of conservation of mass (continuity) and was presented in Chapter II as equation (2.1). The main sources of dissolved oxygen in the stream are atmospheric reaeration and oxygen production by photosynthesis. The major sinks include carbonaceous oxygen demand (CBOD), nitrogenous oxygen demand (NBOD), benthal demand, and respiration of aquatic plants. When these sinks and sources are represented, and all stream system parameters are assumed spatially constant along the flow axis, the governing equation is given by

$$\frac{\partial C}{\partial t} = E \frac{\partial^2 C}{\partial x^2} - U \frac{\partial C}{\partial x} + K_2 (C_s - C)$$

$$- K_1 L - K_n N + P - R_e - B \qquad (7.5)$$

where C = concentration of D.O. in the stream, mg/l,

E = longitudinal dispersion coefficient, ft^2/hr,

U = freshwater stream velocity, ft/hour,

K_2 = atmospheric reaeration coefficient, $hours^{-1}$,

C_s = dissolved oxygen saturation, mg/l,

C_s-C = dissolved oxygen deficit, mg/l = D,

K_1 = deoxygenation constant of carbonaceous BOD, $hours^{-1}$,

L = remaining carbonaceous BOD concentration, mg/l,

K_n = oxidation coefficient of nitrogenous BOD, $hours^{-1}$,

N = remaining nitrogenous BOD concentration, mg/l,

P = oxygen production rate by algal photosynthesis, mg/l-hour,

R_e = algal respiration rate, mg/l-hour, and

B = benthal demand of bottom deposits, mg/l-hour.

This equation assumes no diffusion of pollutants through the river boundaries (other than that which is included in the source-sink term) and is best suited to predict concentrations relatively far downstream from the point of waste injection. Since critical D.O. deficits usually occur several miles

downstream from the waste source, equation (7.5) is particularly well suited for such predictions. For fresh-water streams, the advective flux is significantly larger than the mass flux due to longitudinal dispersion (Hydroscience, 1971). However, it is of interest to present a methodology that can be applied to tidal rivers, and thus longitudinal dispersion is considered. For steady-state analysis, all system parameters are assumed constant in time. Since it is desired to solve for the D.O. deficit and

$$\frac{\partial C}{\partial t} = 0 \;;\; \frac{\partial C}{\partial x} = -\frac{\partial D}{\partial x} \tag{7.6}$$

equation (7.5) reduces to an ordinary differential equation.

$$E \frac{d^2 D}{dx^2} - U \frac{dD}{dx} + K_1 L + K_n N - K_2 D - (P - R_3 - B) = 0 \tag{7.7}$$

A basic assumption of the model, stated earlier, is that the effects of the biological processes ($P - R_e - B$) are incorporated into the measured upstream D.O. deficit. The production and consumption of oxygen by these processes after waste injection are assumed negligible since no data were available. Although measurements of total organic nitrogen and nitrate-nitrogen for upstream sources are available, modeling of the nitrogenous BOD was not possible because

1. The amount of organic and ammonia nitrogen present in all wastewater inputs is unknown, and
2. The oxidation coefficient of nitrogenous BOD is also unknown.

By applying the above assumptions and expressing the remaining carbonaceous BOD, L, in terms of the ultimate carbonaceous BOD and its reaction, the governing differential equation for dissolved oxygen deficit is reduced to the expression

$$E \frac{d^2D}{dx^2} - U \frac{dD}{dx} + K_1 (L_o)_c e^{mx} - K_2 D = 0 \qquad (7.8)$$

where

$$m = \frac{U}{2E} \left[1 - \sqrt{1 + \frac{4K_1 E}{U^2}} \right]$$

$$x \geq 0$$

7.4 Critical Deficit and D.O. Levels

The solution to equation (7.8) as a function of time since release is given by

$$D = \frac{(L_o)_c K_1}{K_2 - K_1} \left[e^{jt} - e^{gt} \right] + D_o e^{gt} \qquad (7.9)$$

where

D = D.O. deficit, mg/l,

K_1 = deoxygenation coefficent, hours^{-1},

K_2 = reaeration coefficient, hours^{-1}, and

t = lapsed time, hours,

$$j = \frac{U^2}{2E}\left[1 - \sqrt{1 + \frac{4K_1E}{U^2}}\right], \text{ hours}^{-1}, \text{ and}$$

$$g = \frac{U^2}{2E}\left[1 - \sqrt{1 + \frac{4K_2E}{U^2}}\right], \text{ hours}^{-1}.$$

To determine the time at which the critical (maximum) deficit occurs, the partial derivative of the D.O. deficit equation (7.9), is taken with respect to time and set equal to zero ($\frac{\partial D}{\partial t} = 0$).

$$0 = \frac{(L_o)_c\ K_1}{K_2 - K_1}\left[je^{jt_c} - ge^{gt_c}\right] + gD_oe^{gt_c} \qquad (7.10)$$

Solving for t_c, the following expression is obtained:

$$t_c = \frac{1}{(j-g)} \quad \left(\frac{g}{j}\right) + \ln\left[\frac{K_1(L_o)_c - D_oK_2 + D_oK_1}{K_1(L_o)_c}\right] \qquad (7.11)$$

Equation (7.11) may be simplified for convenience by making some substitutions,

$$t_c = \frac{1}{(j-g)}\left[\left(\frac{g}{j}\right) + \ln\left(1 - f\,R_o + R_o\right)\right] \qquad (7.12)$$

where

t_c = elapsed time at which the critical deficit occurs, hours,

f = self-purification ratio

= K_2/K_1, and

R_o = ratio of the initial D.O. deficit, D_o to the initial BOD, $(L_o)_c$, dimensionless.

Finally, the critical deficit is found by substituting the value of t_c, given by equation (7.12), into equation (7.9):

$$D_c = \frac{(L_o)_c K_1}{K_2 - K_1} \left[e^{jt_c} - e^{gt_c} \right] + D_o e^{gt_c} \tag{7.13}$$

where

D_c = critical (maximum) deficit, mg/l.

The minimum D.O. level is calculated as

$$C_{min} = C_s - D_c \tag{7.14}$$

where

C_{min} = concentration of D.O. at maximum deficit, mg/l, and

C_s = saturation concenration of D.O., mg/l.

The saturation concentration is determined from the regression relationship (American Society of Civil Engineers, 1960),

$$C_s = 14.652 - 0.41022T + 0.0079910T^2 - 0.000077774T^3 \tag{7.15}$$

where

T = water temperature, °C.

7.5 Reaeration and Deoxygenation Coefficients

The deoxygenation coefficient, K_1, represents the loss of D.O. in the water due to reduction of BOD. A calibrated value of 0.7 day^{-1} is used for this simulation for K_1 at 20°C (68°F). A temperature correction and conversion to hour^{-1} gives

$$K_1(T) = \frac{1}{24} K_1 \ (20°) \ 1.047^{T-20} \tag{7.16}$$

A variety of formulas exists for prediction of the reaeration coefficient K_2, almost all of which depend upon velocity, U, and depth, H. The equation of Langbein and Durum (1967) was chosen because it is most closely related to subsequent procedures used to obtain U and H.

$$K_2 = 2.303 \left[3.3 \frac{U}{H^{1.33}} \right] \tag{7.17}$$

where

K_2 = reaeration coefficient at 20°C, day^{-1},

U = stream velocity, ft/sec, and

H = stream depth, ft.

The problem lies in obtaining values of U and H, since the stream flow varies with time. In the absence of measurements, or if the data cannot be obtained in an expedient manner (as in the ensuing application to the Des Moines River), an approximation can be made based on the

work of Leopold and Maddock (1953) in which they show strong correlations between velocity vs flow and depth vs flow, namely,

$$U = \alpha_1 Q^{\alpha_2} \tag{7.18}$$

$$H = \beta_1 Q^{\beta_2} \tag{7.19}$$

where

Q = streamflow, cfs, and

$\alpha_1, \alpha_2, \beta_1, \beta_2$ = regression coefficients.

The model presently utilizes coefficients which were determined for the Kansas River System in Kansas and Nebraska, for which

$$\alpha_1 = 1.60 \tag{7.20a}$$

$$\alpha_2 = 0.03 \tag{7.20b}$$

$$\beta_1 = 0.11 \tag{7.20c}$$

$$\beta_2 = 0.45. \tag{7.20d}$$

When equations (7.18), (7.19) and (7.20) are substituted into (7.17) and conversion is made to units of hour^{-1}, the reaeration coefficient is established as a function of streamflow, Q,

$$K_2 = 2.303 \, [4.1435 Q^{-0.57}] \, 1.024^{T-20} \tag{7.21}$$

where

T = stream temperature, °C, and the last factor represents a temperature correction.

7.6 Special Problems

During the model application to Des Moines, various problems were encountered revolving around the critical deficit (D_c) and critical time (t_c) equations, equations (7.13) and (7.12), respectively. Due to the large number of conditions being simulated, situations were encountered in which

1. the deficit load ratio, R_o, was undefined

$$R_o = \frac{D_o}{(L_o)_c} \tag{7.22}$$

   because both D_o and $(L_o)_c$ were equal to zero,
2. the self-purification ratio, f, was equal to one, causing equation (7.12) to be undefined, and
3. values of R_o were such that negative values of t_c were obtained.

Mathematical analysis led to the incorporation of certain modifications and safeguards. Thus, equations (7.12) and (7.13) were defined only for

1. $f \neq 1$, or $g \neq j$,
2. $(L_o)_c \neq 0$, and
3. $0 \leq R_o \leq 1/f$.

Otherwise, in order, if

1. $f = 1$, or $g = j$, then

$$D_c = (L_o)_c \, e^{R_o - 1} \tag{7.23}$$

2. $f \neq 1$, $L_o = 0$, then

$$D_c = D_o \tag{7.24}$$

3. $f \neq 1$, $L_o = 0$, $R_o > 1/f$, then

$$D_c = D_o \tag{7.25}$$

These equations, obtained by taking limits, are not particular to Des Moines and are applicable to any receiving stream (Arbabi et al., 1974).

7.7 Application to the Des Moines River

A description of the receiving stream, urban area waste sources, and field measurement efforts has been presented in Chapter III. Methodology has been discussed in Chapters II, III, IV, V and the previous sections of this chapter. Before presenting the results of the receiving water analysis, it is important to discuss model validity. The verification procedure was preceded by calibration of the urban runoff BOD_5 loading rates for Des Moines, Iowa, as computed by STORM. The dust and dirt surface loading factors were adjusted to obtain an annual average BOD_5 concentration of 53 mg/l for urban stormwater runoff. The above concentration was the average value determined by

the field monitoring program in the separate sewer system. The developed mathematical model, as discussed in the methodology, simulates the mixing of stormwater runoff and sanitary sewage in the combined sewer system. The annual average BOD_5 concentration of combined sewer overflows was computed to be 75 mg/l, including the effects of first flush. The average value determined by the field monitoring program in the combined sewer system was determined to be 72 mg/l (Davis and Borchardt, 1974).

Verification Analysis

An important part of the total effort required to develop a mathematical model of water quality in a stream is devoted to verification and improvement of model accuracy. The verification procedure recommended for steady-state water quality models includes

1. examination of model output using preliminary coefficients on a diverse set of data (different waste loads and temperatures under conditions of high and low flow, and variable initial stream quality);
2. assessment of the closeness of fit of observed field data to computed values;
3. adjustment of the model coefficients until the desired accuracy is obtained; and

4. achievement of a mathematical abstraction that reasonably reproduces observed stream response and establishes the necessary validity for planning purposes.

The carbonaceous BOD reaction coefficient, K_1, was refined during the verification process to a final value of 0.70 day^{-1} (at 20°C). The model, of course, converts to units of $hour^{-1}$ and adjusts for temperature through equation (7.16). The atmospheric reaeration coefficient, K_2, is calculated internally as a function of streamflow and temperature by equation (7.21); therefore, no adjustment was necessary. Since field-measured stream dissolved oxygen concentrations were given as daily values (Davis and Borchardt, 1974), storm-averaged D.O. concentrations were determined. Measured daily values and storm-average computed values of D.O. at a point 5.6 miles (9.0 km) downstream from the confluence of the Raccoon and Des Moines Rivers are compared in Figure 7-1. Correlation between the calculated and observed profiles is quite good. The point corresponds to sampling location No. 6 as shown previously in Figure 3-2.

Included in Figure 7-1 are rainfall and average total river flow values for each wet weather event (as defined in Figure 4-3). Differences between measured and computed D.O. concentrations may be attributed to such factors as (1) the time of day during which the sample was taken, (2) the lag time between sampling and laboratory analysis and the temperature variations in the receiving water during the day,

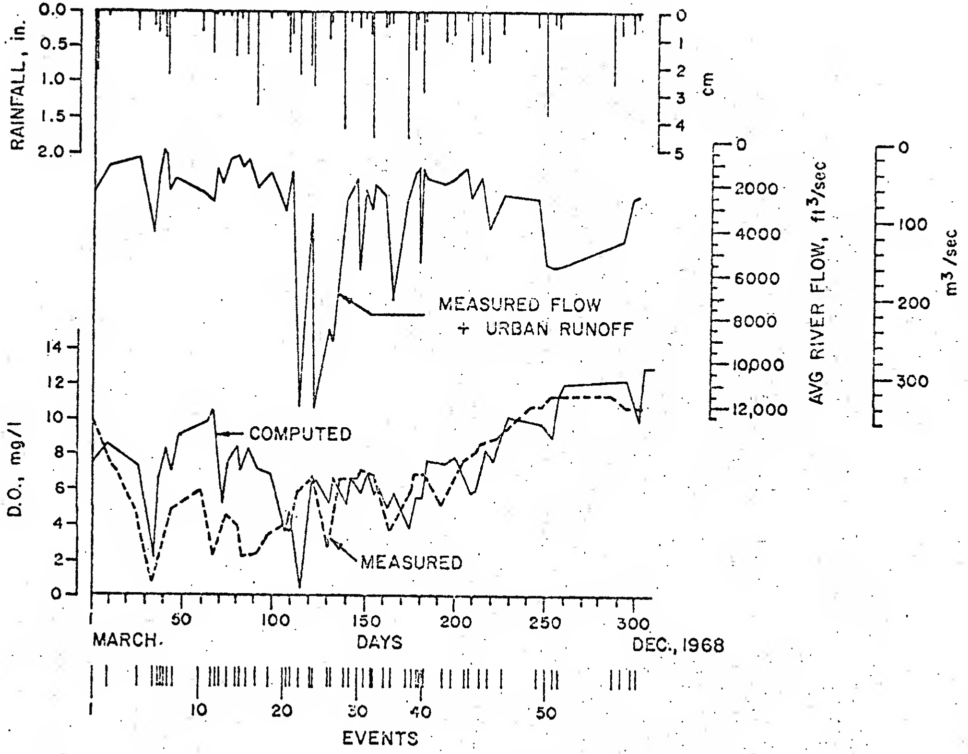


Figure 7-1. Application to Des Moines, Iowa. Measured and Computed Values of D.O. at 5.6 mi (9.0 km) downstream from confluence of Raccoon and Des Moines Rivers.

and (3) a lack of data on photosynthesis, algal respiration, and benthic demand. The time scale in days represents the wet year beginning on March 8 and ending December 30, 1968. Again, it should be emphasized that these D.O. values are not the minimum D.O's resulting from maximum deficits. The maximum deficits occur much further downstream and water quality standards are violated much more frequently.

A longitudinal dispersion coefficient, E, of 50 ft^2/sec (16,722 sq m/hour) was assumed for the Des Moines River. Verification was not possible by field measurement. However, its order of magnitude is within the range of a moderate, free-flowing freshwater stream (Hydroscience, 1971). For such flow regimes, the advective flux is much larger than the dispersive mass flux. Nevertheless, a stretch of river may undergo a transition from a freshwater stream with a high velocity (during the high flow season) to a semi-tidal river with low velocity during the low flow period (Hydroscience, 1971). It is important to compile data on longitudinal dispersion for large, deep rivers where the effect of dispersive mass flux can generally not be neglected for time-varying waste inputs (Thomann, 1973).

Results

A review of Table 7-1 is appropriate at this stage. The urban waste load to the receiving stream may be significantly controlled by the community itself through

(1) the treatment rate given municipal wastewaters at the DWF plant, and (2) the size of the WWF storage/treatment facility, which determines the residence time of urban stormwater inflows. Receiving water response is given for the combined effects of urban waste sources and upstream waste sources. Essentially by varying the degree of control, as shown in Table 7-1, 10 different combinations of waste inputs are investigated for the stretch of the Des Moines River extending downstream from its confluence with the Raccoon River (refer to Figure 3-1). In addition, the response of the receiving water is shown for the unlikely situation that no treatment whatsoever is provided by the urban environment, as a basis for comparison.

Results are presented in the form of minimum dissolved oxygen cumulative frequency curves, as shown in Figures 7-2 and 7-3. The ordinates represent the percentage of critical D.O. concentrations exceeding the given stream standard (abscissa) during the wet weather period. The critical (minimum) D.O. concentration in the stream is computed for every hour of wet weather during the year. Thus, the ordinates also represent the percentage of all wet weather hours that exceed a particular dissolved oxygen level. The dashed curve labeled NO TREATMENT in both figures represents the receiving water response for the unique case in which

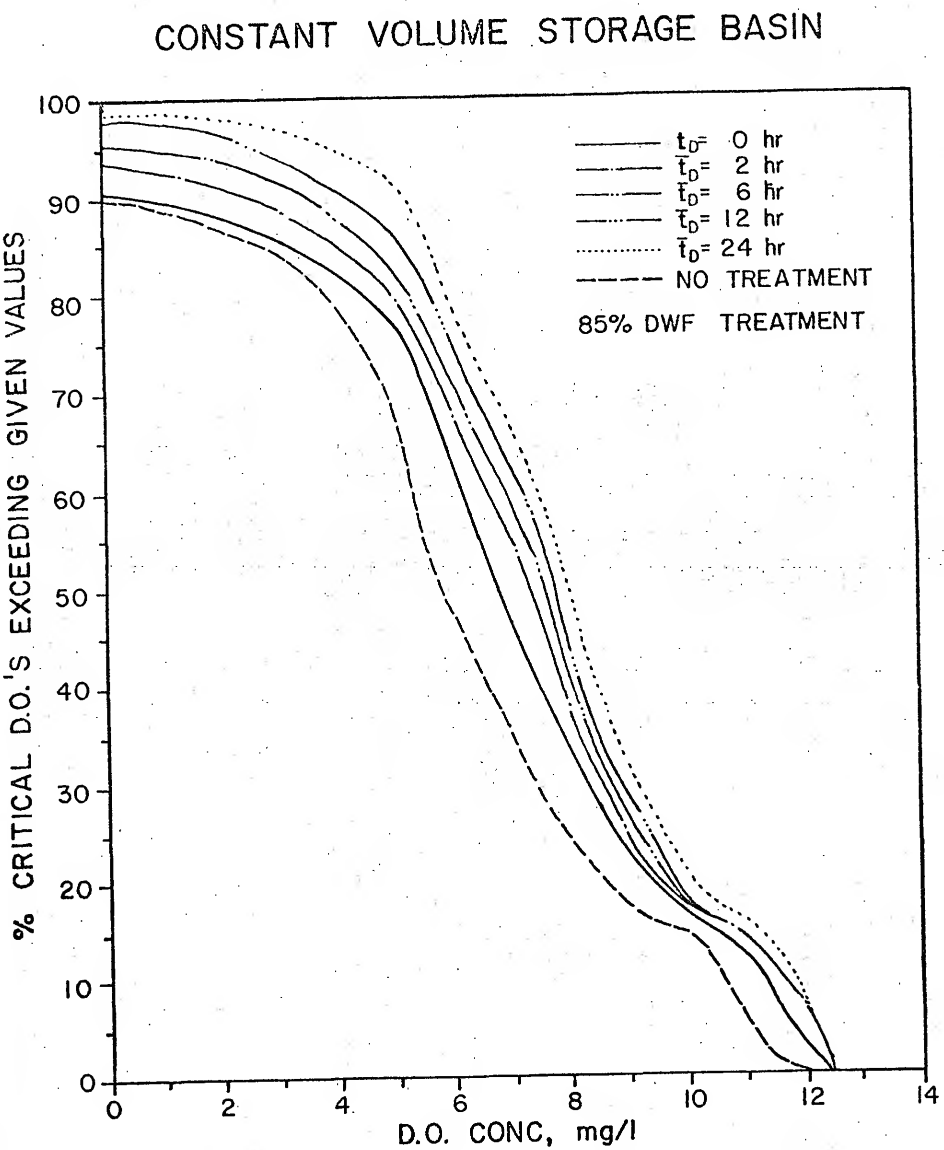


Figure 7-2. Minimum D.O. Frequency Curves for Varied WWF Control and DWF Secondary Treatment Versus No Treatment of Urban Waste Sources.

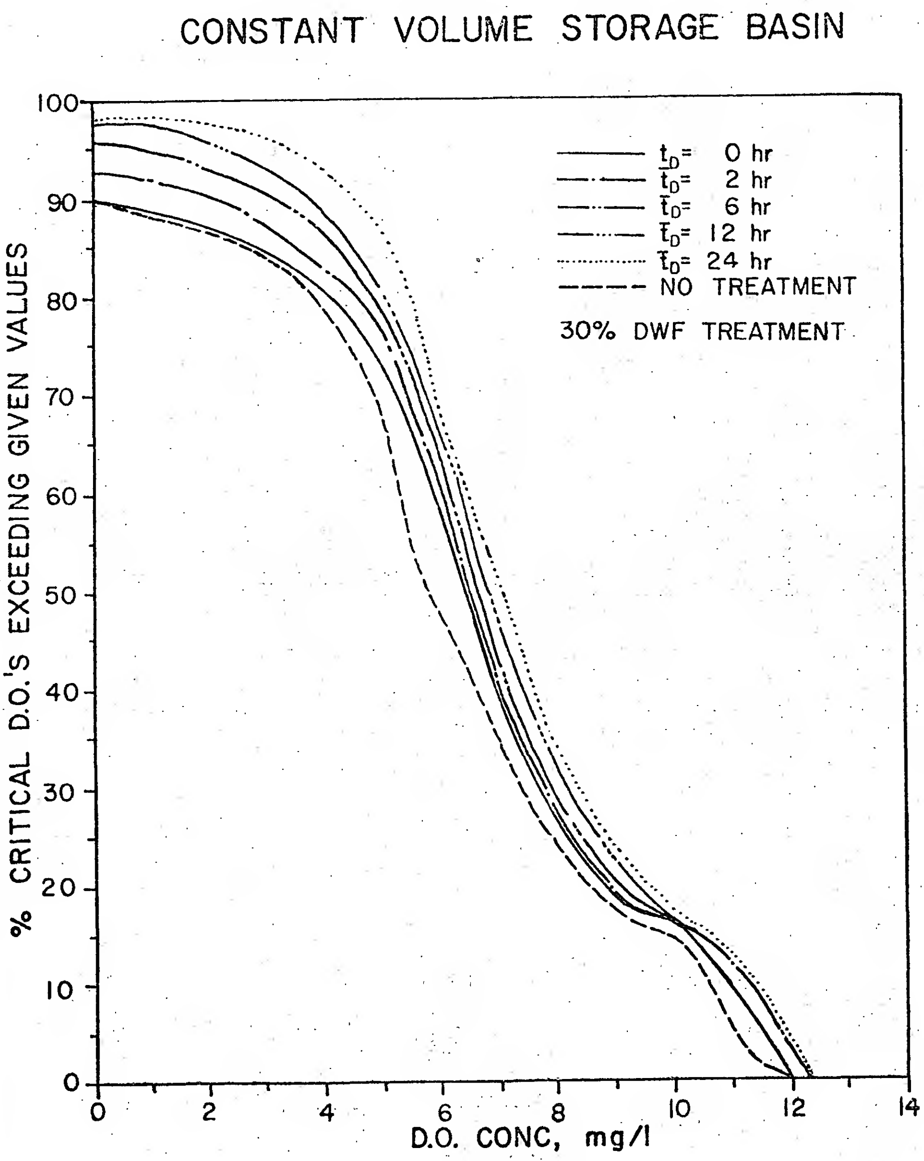


Figure 7-3. Minimum D.O. Frequency Curves for Varied WWF Control and DWF Primary Treatment Versus No Treatment of Urban Waste Sources.

neither the municipal wastewaters nor the urban stormwater flows are given any degree of control. The State of Iowa enforces a dissolved oxygen stream standard which consists of (1) maintaining a minimum dissolved oxygen concentration of at least 4.0 mg/l at all times, and (2) maintaining a D.O. concentration of at least 5.0 mg/l for at least 16 hours a day (State Hygienic Laboratory, 1970). From Figures 7-2 and 7-3, it is possible to determine the percentage of hourly minimum D.O. concentrations that meet the required levels for all wet weather events during the entire period of simulation, March 8 to December 30, 1968. There was a total of 449 hours of wet weather during this period, producing a total runoff of 10.28 inches (261 mm).

It can be concluded from Figures 7-2 and 7-3 that changing the DWF treatment rate from secondary to primary reduces the incremental benefit derived from maintaining higher detention times in the WWF storage/treatment facility, especially at the higher D.O. levels. However, it is also clear that receiving water response is much more sensitive to variations in the WWF storage/treatment detention times. The effectiveness of WWF control, while providing secondary treatment to DWF, is summarized in Table 7-2 and compared to the option of no control of urban wastewaters. It appears that if the goal of maintaining an absolute minimum D.O. level of 4.0 mg/l in

Table 7-2

Percentage of Critical D.O. Concentrations Exceeding Given Stream Standard*

Degree of Control	Stream Standard of 4.0 mg/l	Stream Standard of 5.0 mg/l
Well-mixed Constant Volume WWF Storage/ Treatment		
1. $\bar{t}_D$ = 0, 85% DWF Treatment	82%	77%
2. $\bar{t}_D$ = 2 hrs, 85% DWF Treatment	86	80
3. $\bar{t}_D$ = 6 hrs, 85% DWF Treatment	88	82
4. $\bar{t}_D$ = 12 hrs, 85% DWF Treatment	91	86
5. $\bar{t}_D$ = 24 hrs, 85% DWF Treatment	95	91
No Control of Urban Wastewaters	77	68

*During period of wet weather.

the stream is to be achieved, a 24-hour detention facility will be needed. For practical reasons, it is likely that an urban community would have to build a number of these facilities rather than a very large central holding basin. Nevertheless, the results shown in Figures 7-2 and 7-3 and Table 7-2 indicate the need for the higher detention times in the WWF control facilities. It is anticipated that further research into storage/treatment of urban stormwater will be needed, but results indicate that if a storage facility maintains adequate detention of wet weather flows, equalization and first-order decay effects are sufficient to provide the degrees of control to prevent extended violations of receiving water D.O. standards.

It is interesting to note that the mathematical model predicted stream velocities in the Des Moines River ranging from 1.9 to 2.1 ft/sec (0.58 to 0.64 m/sec). At these velocities, it can be shown that the advective flux is more significant than the dispersive flux by several orders of magnitude. At the lower levels of WWF control, critical distances up to about 50 miles (80 km) were predicted. By contrast, at the higher levels of WWF control critical distances up to about 20 miles (32 km) were predicted.

Unquestionably, a detention facility for urban stormwater flows has a favorable impact on receiving water

quality. The Humboldt Avenue combined sewer overflow detention tank (City of Milwaukee _et al_., 1975), constructed as a demonstration project to intercept overflow from a 750-acre (231-ha) segment of the urban area, provides a good example. The detention facility had a capacity of 3.9 million gallons (14,762 cubic meters). An analysis of the data obtained during the 12-month period between November 1, 1971, and October 31, 1972, indicated that the project facility prevented approximately 100,000 pounds of BOD (45,359 kg BOD) out of 147,000 total pounds of BOD (66,678 kg BOD) from being discharged to the Milwaukee River (City of Milwaukee _et al_., 1975). This indicates a percentage removal efficiency of 68 percent for BOD. The Humboldt Avenue detention tank may be compared to a constant volume storage/treatment system providing approximately 20 hours of detention. Direct effects on river quality, however, could not be assessed adequately due to several factors exerting significant but undetermined influence, such as (1) algal and macrophyte activity, and (2) benthal oxygen demand from accumulated sediments (City of Milwaukee _et al_., 1975).

CHAPTER VIII

SUMMARY AND CONCLUSIONS

A unified concept of pollutant mass transport through the various components of the physical system has been presented. Mathematical models derived from the unifying principle of continuity have been developed and successfully applied to an actual urban area configuration (Des Moines, Iowa) and its receiving water. Deterministic and statistical techniques have been blended to serve complementary roles. Thus, the interaction of urban runoff, control measures and receiving water response has been demonstrated. Since continuous simulation was performed for all wet weather hours during the precipitation year of record, frequency analyses of BOD concentrations and mass rates into and from the WWF storage/treatment systems, as well as of minimum D.O. concentrations in the receiving stream resulting from these BOD inputs, were possible.

Although the product Kt_D may be a useful indicator of treatment potential in the various components of the urban and natural environments (e.g., sewer pipe segments, DWF treatment plants, WWF detention facilities,

rivers, lakes, and estuaries), the detention time t_D has been shown to be the most critical parameter in storage/treatment systems subjected to urban runoff inflows. Mathematical models were developed for a well-mixed constant volume system, a well-mixed variable volume system, and a dispersive variable volume system. In all cases, significant reductions in peak and mean BOD loads to the receiving body of water were predicted at the higher levels of detention (6 to 24 hours). Results obtained from the well-mixed constant volume model compared closely to the performance of the Humboldt Avenue combined sewer overflow detention tank in Milwaukee, Wisconsin.

The receiving water response was evaluated, in terms of critical deficits and resulting minimum D.O. concentrations, for the case where the WWF storage/treatment system was represented by the well-mixed constant volume model. The results indicated that the WWF control device was effective in improving minimum D.O. levels in the stream for average detention times of 2, 6, 12, and 24 hours. From inspection of the frequency analyses of BOD concentrations and mass rates into and from the other storage/treatment systems modeled, and the pollutant removal efficiencies predicted, it appears reasonable to conclude that the variable volume devices would perform equally well or better.

Several recommendations for further research are in order.

(1) Additional modeling needs include a more detailed representation of DWF treatment processes, and extension of the receiving water analysis to the variable volume WWF storage/treatment systems;

(2) Concepts from autocorrelation analysis and the correlogram may be useful to further characterize the response of the different storage/treatment systems; and

(3) More extensive verification of the performance of these storage/treatment systems may be possible if demonstration projects are financed to construct prototypes in other urban areas.

APPENDICES

APPENDIX I

WELL-MIXED AND DISPERSIVE VARIABLE VOLUME DIGITAL COMPUTER MODEL

```
STOR:PROC OPTIONS(MAIN)   ;
      DCL 1 DATA2,
          2 RR1,
            (3 RRR1,3 RRR2,3 RRR3) FLOAT,
          2 RR2,
            (3 RRR4,3 RRR5,3 RRR6,3 RRR7,3 RRR8,3 RRR9,3 RR10) FLOAT;
      DCL 1 DATA CONTROLLED,
          2 READ1,
            (3 FLOWX,3 BODMX,3 BODCX) FLOAT,
          2 READ2,
            (3 DATE,3 DWHX,3 HOUR,3 Q,3 RBOD,3 RT,3 RDO) FLOAT;
      DCL (DIS,MIT) FLOAT;
      DCL (X1,X2,X3,X4,Y1,Y2,Y3,Y4,X5,Y5) FLOAT;
      DCL (XM(58),YM(58),XS(58),YS(58),CVX(58),CVY(58)) FLOAT;
      DCL (FLOW1(100),BODM1(100),BODC1(100)) FLOAT;
      DCL (TDET(58),XCFWM(58),XCCFWM(58),ASADT,VMAX) FLOAT;
      DCL (Z1,Z2,Z3,ZZ4,ZZ5) FLOAT;
      DCL (ZM(58),ZS(58),CVZ(58)) FLOAT;
      DCL (KS,V,K1,ST,RR,RELRATE) FLOAT;
      DCL (XC1,XC2,XC3,XCM(58),XCS(58),CVC(58)) FLOAT;
      DCL (KIT,ATOT,ASEP,ACUM) FLOAT;
      DCL (PCTTRT(3),TD(80),PCT(3),KSTO,KST(80),DWFLO) FLOAT;
      DCL (DWF,X,XC(80,3,3),DO(80),FLO(80),DC(80,3,3)) FLOAT;
      DCL (DOCONC(80,3,3),TE(80),DT(80,3,3),DXCONC(80,3,3)) FLOAT;
      DCL (TC(80,3,3),F(80),LO(80,3,3),K2T(80),U(80),VS(80)) FLOAT;
      DCL (QSEP(80),QCOMB(80)) FLOAT;
      DCL (PBODCOMB(80),CONCCOMB(80),COMBLO(80)) FLOAT;
      DCL (LOC(80,3,3),MASSIN(80),CINSTO(80),COUTSTO(80)) FLOAT;
      DCL (SEPLO(80),MASSOUT(80),CSFLO(80),DATA1(2500)) FLOAT;
      DCL (DATA3(2500),CMAX,CMAX2,DATA4(2500)) FLOAT;
      DCL (TTIME(80),FSTOR(80),CIN(80)) FLOAT;
      DCL (NN,N,NW) FIXED DECIMAL (4);
      DCL (ALPHA,VV,VOL(80),AVOL(80),QO(80),QOCFS(80)) FLOAT;
```

```
      DCL  (XFM(58),KSED(58),XFS(58),CVF(58),ADT,TSET,XF1,XF2,XF3) FLOAT;
      DCL  (VOLNN,QINIT,INITVOL,QOTO(58),DWT(58),VOLTO) FLOAT;
      DCL  (XFFM(58),XFFS(58),CVFF(58),XFF1,XFF2,XFF3) FLOAT;
      DCL  (XCC1,XCC2,XCC3,XCCM(58),XCCS(58),CVCC(58)) FLOAT;
      DCL  (INITCONC,CONCTO,CNCTO(59),CONCNN(59)) FLOAT ;
      DCL  (YMD(58),YMMI(58),YSD(58),YSSI(58),CVM(58),CVMI(58)) FLOAT;
      DCL  (IMFLOW,IMBODM,IMBODC,IBODCCEV,IFLOWDEV,IBODMDEV) FLOAT;
      DCL  (ACOUTSTO(80),QCSTO(80),AMASSOUT(80),AQOCFS(80)) FLOAT;
      DCL  (FT(120),T1,T(120,6)) FLOAT(9);
      DCL  (MK1,RK2,AX,UST,KX,OMEGA,LST,ES,ER) FLOAT(9);
      DCL  (SUM1,YM1,YM2,YM3) FLOAT ;
      DCL  (XMSEP,QMAX,XMCOM,XMSFL,XMCFL,AVBODSEP,AVBODCOM) FLOAT ;
            DATA1,DATA3,DATA4=0.0 ;
      GET LIST(INITVOL,INITCONC,ALPHA,VV,RR,RELRATE)   ;
      GET LIST(N,MIT,NW,V,K1,KS,ST)   ;
      GET LIST(DIS,T1,LST,ES,ER)   ;
      GET LIST(ATOT,ASEP,ACOM,PDWFBODX,DWF)   ;
      GET LIST(PCTTRT,DWFLO,X,PCT)   ;
      GET LIST(FFLBS)   ;
      OPEN FILE(BOD) STREAM INPUT;
      OPEN FILE(PRINT1) STREAM OUTPUT;
      OPEN FILE(PRINT2) STREAM OUTPUT;
      PUT PAGE ;
      PUT SKIP(5) EDIT('*** VOLUME =',V,' C.F.*** ALPHA =',ALPHA,' ***')
                        (A,F(12,2),A,F(7,4),A) ;
      PUT SKIP(2) EDIT('*** VV =',VV,' *** RR =',RR,' *** DIS =',DIS,' *
**') (A,F(5,2),A,F(5,2),A,F(5,2),A) ;
      PUT SKIP(2) EDIT('*** ST =',ST,' *** KS =',KS,' 1/HOUR *** ES =',E
S,' FT**2/HOUR *** LST =',LST,' FT ***') (A,F(5,2),A,F(7,3),A,F(9,2),A,
F(8,2),A) ;
      IC,ID,IL,X1,Y1,X2,Y2,X3,Y3,X4,X5,Y4,Y5=0.0 ;
      XI4,XI5,YI4,YI5,ZZI4,ZZI5,W4,WI4=0.0 ;
      XCVF,XCVFI,XCVM,XCVMI,XCVC,XCVCI=0.0 ;
```

```
FLOW1=0 ; BODM1=0 ; RBODM=0 ; RFLOW=0 ;
VCLTO=INITVCL ; DWT=0.0 ; QOTO=0.0 ;
CONCTO=INITCONC ; CNCTO=0.0 ;
BODC1=0 ; Z1,Z2,Z3,ZZ4,ZZ5=0 ;
PDWFBOD=0.0 ; TDET=0.0 ; QSEP=0.0 ; QCCMB=0.0 ;
PCTTRT=1.-PCTTRT ; ASADT,VMAX=0.0 ;
OCSTO=0.0 ;
CMAX,QMAX,CMAX2=0.0 ;
PCT=1.-PCT ; ACT=1./ALPHA ; D=X/5280.0 ;
     IY=0 ; IX=0 ; XMSEP,XMCOM,XMSFL,XMCFL=0.0 ;
         /** DISPERSION FUNCTION F(T-TAU) **/
 IF DIS=1.0 THEN DO ;
 UST=LST*ALPHA ; S2,S3,S4,S5,S6=0.0 ;
 OMEGA=SQRT(UST*UST+4.0*KS*ES) ;
 DO IF=1 TO 100 ;
 FT(IF)=0.0 ;
 DO IT=1 TO 6 ;
 T(IF,IT)=DECIMAL(IF-1,6,2)+DECIMAL(IT-1,6,2)*0.20 ;
 END ;
 FT(IF)=(FNFT(T(IF,1),T1)/2.0+FNFT(T(IF,2),T1)+FNFT(T(IF,3),T1)+FN
FT(T(IF,4),T1)+FNFT(T(IF,5),T1)+FNFT(T(IF,6),T1)/2.0)/5.0 ;
 IF IF=1 THEN FT(IF)=FNFT(T(IF,6),T1) ;
 END ;
   FNFT: PROCEDURE (TF,TAU) RETURNS(FLOAT(9)) ;
         DCL (TF,TAU,S2,S3,S4,S5,S6,CTFT) FLOAT(9) ;
          S4=LST*OMEGA/(2.0*ES) ;
          IF TF¬=0.0 THEN DO ;
         S2=(LST+OMEGA*TF)/SQRT(4.0*ES*TF) ;
         S5=(LST-OMEGA*TF)/SQRT(4.0*ES*TF) ;
          END ;
         IF TF>TAU THEN DO ;
         S3=(LST+OMEGA*(TF-TAU))/SQRT(4.0*ES*(TF-TAU)) ;
         S6=(LST-OMEGA*(TF-TAU))/SQRT(4.0*ES*(TF-TAU)) ;
```

```
                CTFT=EXP(LST*UST/(2.0*ES))*((ERF(S2)-ERF(S3))*EXP(S4)-(ER
F(S5)-ERF(S6))*EXP(-S4)) ;
                RETURN(CTFT) ; END ;
                IF TF=TAU THEN DO ;
                CTFT=EXP(LST*UST/(2.0*ES))*((ERF(S2)-1.)*EXP(S4)-(ERF(S5)
-1.)*EXP(-S4)) ;
                RETURN(CTFT) ; END ;
                    ELSE RETURN(0.0) ;
                END FNFT ;
      END ;
    /* STATISTICS OF INDEPENDENT EVENTS */
     J=1 ; IN=0 ; IX=C ; IY=0 ;
EVNT: I=1 ;
      IE=1 ;
  RD: ALLOCATE DATA;
     GET FILE(BOD) EDIT(READ1) (F(9,2),F(9,1),F(9,1));
     GET LIST(READ2);
     IC=IC+1 ; IL=IL+1 ;
     IF IL=1 THEN DWT(IL)=DWHX ;
     IF DWHX>MIT & IL¬=1 THEN DO ;
        IE=IE+1 ;
        DWT(IE)=DWHX ;
        GO TO CNT ;
          END ;
    /* INSERTING ZERO MASS AND FLOW RATE FOR DW HOURS */
                /* WITHIN RUNOFF EVENTS */
     IF IL=1 THEN GC TO L2 ;
     IF IC=1 THEN GC TO L2;
     IF DWHX=0 THEN GO TO L2 ;
     ICWH=DWHX ;
     DO II=1 TO IDWH ;
     IS=II+J-1 ;
     FLOW1(IS)=0 ; GSEP(IS)=0.0 ; GCOMB(IS)=0.0 ;
```

```
     BODM1(IS)=0 ;
     BODC1(IS)=0 ;
     END ;
     J=J+IDWH ;
     IN=IN+IDWH ;
 L2:  FLOW1(J)=FLOWX;
     BODM1(J)=BODMX;
     BODC1(J)=BODCX;
     QSEP(J)=FLOW1(J)*ASEP/ATOT ;
     QCOMB(J)=FLOW1(J)*ACOM/ATOT ;
             /* END */
     IF IC¬=1 THEN ID=ID+DWHX;
     X1=X1+FLOW1(J) ; Y1=Y1+BODM1(J) ; Z1=Z1+BODM1(J)*4.45 ;
     X2=X2+FLOW1(J)*FLOW1(J) ; Y2=Y2+BODM1(J)*BODM1(J) ;
       J=J+1 ; IN=IN+1 ;
     IF IL=342 THEN GO TO CNT;
     GO TO RD ;
CNT: DATA2=DATA ;
     DO II=1 TO IC-1 ; FREE DATA ; END ;
     IF IL=342 THEN IC=IC+1 ;
     NN=IC+ID-1 ;
        XM(I)=X1 ; YM(I)=Y1 ;
        IF VV¬=1.0 THEN TDET(I)=V/(X1*3600.0) ;
        XS(I)=0 ; YS(I)=0 ;
        ZM(I)=Z1/X1 ;
     IF NN=1 THEN GO TO S ;
     XM(I)=X1/NN ; YM(I)=Y1/NN ;
     ZM(I)=Z1/X1 ;
     X3=XM(I)*XM(I)/(NN-1) ; Y3=YM(I)*YM(I)/(NN-1) ;
     XS(I)=SQRT(X2/(NN-1)-NN*X3) ; YS(I)=SQRT(Y2/(NN-1)-NN*Y3) ;
     IF VV¬=1.0 THEN DO ; TDET(I)=V/(XM(I)*3600.0) ;
     ASADT=ASADT+TDET(I) ; END ;
  S: CVX(I)=0 ; CVY(I)=0 ;
```

```
      IF XM(I)¬=0 THEN CVX(I)=XS(I)/XM(I) ;
      IF YM(I)¬=0 THEN CVY(I)=YS(I)/YM(I) ;
              IF I=52 THEN DO ;
      PUT PAGE ;
      PUT SKIP(3) EDIT('***** RUNOFF EVENT NO.=',I,' *****') (A,F(5),A);
      PUT SKIP(2) LIST('***** FLOW RATE INPUT , CFS *****');
      PUT EDIT((FLOW1(IJ) DO IJ=1 TO NN)) (SKIP,(10)F(9,1));
      PUT SKIP(2) LIST('***** BOD MASS RATE INPUT , LBS/HOUR *****');
      PUT EDIT((BODM1(IJ) DO IJ=1 TO NN)) (SKIP,(10)F(9,1));
      PUT SKIP(2) LIST('***** BOD CONC. , MG/L *****');
      PUT EDIT((BODC1(IJ) DO IJ=1 TO NN)) (SKIP,(10)F(9,1));
      PUT SKIP(2) EDIT('*** XM =',XM(I),' CFS *** YM =',YM(I),' LBS/HR *
**') (A,F(9,1),A,F(9,1),A);
      PUT SKIP EDIT('*** XS =',XS(I),' CFS *** YS =',YS(I),' LBS/HR ***'
) (A,F(9,1),A,F(9,1),A);
      PUT SKIP EDIT('*** CVX =',CVX(I),' *** CVY =',CVY(I),' ***')
(A,F(5,2),A,F(5,2),A);
      PUT SKIP(2) LIST('*** FLOW WEIGHTED STORMWATER BOD CONC. ***') ;
      PUT SKIP(2) EDIT('*** ZM =',ZM(I),' MG/L ***') (A,F(9,1),A);
      NC=IC-1 ;
      PUT SKIP(3) EDIT('*** IL =',IL,' *** NC =',NC,' *** NN =',NN,' ***
') (A,F(4),A,F(3),A,F(3),A);
      PUT SKIP EDIT('*** IN =',IN,' ***') (A,F(4),A);
              END ;
      XC1,XC2,XC3,XF1,XF2,XF3=0.0 ;
      XCC1,XCC2,XCC3,XFF1,XFF2,XFF3,XCFW1,XCCFW1=0.0 ;
      YM1,YM2,YM3,YMM1,YMM2,YMM3=0.0 ; SUM1=0.0 ;
 URB: DO IR=1 TO NN ;
      DWFSEP=ASEP*DWF/ATOT ;
      DWFCOMB=ACOM*DWF/ATOT ;
      SEPLO(IR)=BODC1(IR) ;
      IF ASEP=0.0 THEN SEPLO(IR)=0.0 ;
      PBODCOMB(IR)=BODC1(IR)*QCOMB(IR)*0.2248+ACOM*PDWFBODX/ATOT ;
```

```
      CSFLO(IR)=QCOMB(IR)+DWFCOMB ;
      FF=0.0 ; CONCCOMB(IR)=0.0 ;
      IF CSFLO(IR)¬=0.0 THEN DO ;
      FF=FFLBS*ACOM*DWT(I) ; IF IR¬=1 THEN CONCCOMB(IR)=PBODCOMB(IR)/(CS
FLO(IR)*3600.0) ; IF IR=1 THEN CONCCOMB(IR)=(PBODCOMB(IR)+FF)/(CSFLO(IR
)*3600.0) ;
      END ;
      CCMBLO(IR)=CONCCOMB(IR)*16016.60 ;
      XMSEP=XMSEP+SEPLO(IR)*QSEP(IR) ; XMSFL=XMSFL+QSEP(IR) ;
      XMCOM=XMCOM+COMBLO(IR)*CSFLO(IR) ; XMCFL=XMCFL+CSFLO(IR);
         /* STORAGE OF STORM SEWER FLOWS AND COMBINED */
               /* SEWER OVERFLOWS */
         /* VARIABLE VOLUME -- DISCRETE INPUTS */
      S1,S2=0.0 ;
      CINSTO(IR)=0.0 ; COUTSTO(IR)=0.0 ;
      FSTOR(IR)=0.0 ;
      FSTOR(IR)=QSEP(IR)+QCOMB(IR) ;
      IF FSTOR(IR)¬=0.0 THEN DO ;
      CINSTO(IR)=(QSEP(IR)*BODC1(IR)+QCOMB(IR)*CCMBLO(IR))/FSTOR(IR) ;
      END ;
      IF DIS=1.0 THEN QCSTO(IR)=UST*CINSTO(IR)/(2.0*OMEGA) ;
         /* MG/L =4.45 LBS/HR/CFS */
      MASSIN(IR)=FSTOR(IR)*CINSTO(IR)*0.224719 ;
      END URB ;
      KSED(I)=0.0 ; IF I¬=1 THEN DO ; TSET=0.0 ;
      IF VV=1.0 & ALPHA>0.1 THEN TSET=DWT(I)-1.0/ALPHA ;
      IF TSET<0.0 THEN TSET=0.0 ; IF TSET>20.0 THEN TSET=20.0 ;
      KSED(I)=-LOG(0.90+0.10*EXP(-0.7*TSET)) ; END ;
      CNCTO(I)=CONCTO*EXP(-KS*DWT(I)-KSED(I)) ;
 VAR: DO IR=1 TO NN ;
       CO(IR)=0.0 ; ACOUTSTO(IR)=0.0 ;
       COTO(I)=ALPHA*VOLTO*EXP(-ALPHA*DWT(I)) ;
         IF RR=1.0 THEN DO ;
```

```
      QOTO(I)=ALPHA*(VOLTO-RELRATE*DWF*3600.0*DWT(I)) ;
      IF QOTO(I)<0.0 THEN QOTO(I)=0.0 ;
        END ;
     CINIT=QOTO(I)/3600.0 ;
     CO(IR)=FSTOR(IR)*3600.0-(FSTCR(IR)*3600.0-QOTO(I))*EXP(-ALPHA) ;
     IF IR-1¬=0 THEN DO ;
     CO(IR)=FSTCR(IR)*3600.0-(FSTCR(IR)*3600.0-CO(IR-1))*EXP(-ALPHA) ;
     END ;
     VOL(IR)=QO(IR)/ALPHA ;
     QOCFS(IR)=QO(IR)/3600.0 ;
     KSTO=-LOG(0.90+0.10*EXP(-0.70*ADT)) ;
     S1=ALPHA+KS+KSTO ;
     S2=ALPHA*FSTOR(IR)*3600.0*CINSTO(IR)/(S1*QO(IR)) ;
     S3=QOTO(I)/QO(IR) ; IF IR-1¬=0 THEN S3=QO(IR-1)/QO(IR) ;
     COUTSTO(IR)=S2*(EXP(S1)-1)*EXP(-S1)+S3*CNCTO(I)*EXP(-S1) ;
     MASSOUT(IR)=(0.224719*ALPHA*FSTOR(IR)*CINSTO(IR)/S1)*(1-EXP(-S1))
+0.224719*QINIT*CNCTO(I)*EXP(-S1) ;
     IF IR-1 ¬=0 THEN DO ;
     COUTSTO(IR)=S2*(EXP(S1)-1)*EXP(-S1)+S3*COUTSTO(IR-1)*EXP(-S1);
     MASSOUT(IR)=(0.224719*ALPHA*FSTOR(IR)*CINSTO(IR)/S1)*(1-EXP(-S1))
+0.224719*QOCFS(IR-1)*COUTSTO(IR-1)*EXP(-S1) ;
     END ;
     IF COUTSTO(IR)<0.0 THEN COUTSTO(IR)=0.0 ;
     AQOCFS(IR)=(FSTOR(IR)*3600.0+((FSTOR(IR)*3600.0-QOTO(I))/ALPHA)*(
EXP(-ALPHA)-1.))/3600.0 ;
     AMASSOUT(IR)=(0.000062/S1)*(S2*S1*QO(IR)+(S2*QO(IR)-QOTO(I)*CNCTO
(I))*(EXP(-S1)-1.)) ;
     IF AQOCFS(IR)¬=0.0 THEN ACOUTSTO(IR)=AMASSOUT(IR)*4.45/AQOCFS(IR)
     ; IF IR¬=1 THEN DO ;
     AQOCFS(IR)=(FSTOR(IR)*3600.0+((FSTOR(IR)*3600.0-QO(IR-1))/ALPHA)*
(EXP(-ALPHA)-1.))/3600.0 ;
     AMASSOUT(IR)=(0.000062/S1)*(S2*S1*QO(IR)+(S2*QO(IR)-QO(IR-1)*COUT
STO(IR-1))*(EXP(-S1)-1.)) ;
```

```
      IF AQOCFS(IR)¬=0.0 THEN ACOUTSTO(IR)=AMASSOUT(IR)*4.45/AQOCFS(IR)
      ; END ; IF ACOUTSTO(IR)<0.0 THEN ACOUTSTO(IR)=0.0 ;
      IF I=1 THEN CONCNN(I)=INITCONC ;
        CONCNN(I+1)=COUTSTO(IR) ;
      VOLNN=VOL(IR) ;
      AVOL(IR)=AQOCFS(IR)*3600.0/ALPHA ;
      IF AVOL(IR)>VMAX THEN VMAX=AVOL(IR) ;
      IF AQOCFS(IR)>QMAX THEN QMAX=AQOCFS(IR) ;
      END VAR ;
 DWP: DO IR=NN+1 TO NW ;
      QO(IR)=QO(IR-1)*EXP(-ALPHA) ;
      QOCFS(IR)=QO(IR)/3600.0 ;
       AQOCFS(IR)=-QOCFS(IR-1)*(EXP(-ALPHA)-1.)/ALPHA ;
       ACOUTSTO(IR)=ACOUTSTO(IR-1)*EXP(-KS) ;
       AMASSOUT(IR)=ACOUTSTO(IR)*AQOCFS(IR)*0.224719 ;
      END DWP ;
         /** CONVOLUTION **/
DISP: IF DIS¬=1.0 THEN GO TO P1 ;
      NNZ=0 ;
      DO JT=1 TO NW ;
      SUM1=0.0 ;
      DO IR=1 TO NN WHILE(JT>IR) ;
      SUM1=SUM1+QCSTO(IR)*FT(JT-IR) ;
      END ;
      ACOUTSTO(JT)=SUM1 ;
      AMASSOUT(JT)=AQOCFS(JT)*ACOUTSTO(JT)*0.224719 ;
      IF ACOUTSTO(JT)>0.1 THEN DO ;
         NNZ=NNZ+1 ;
      IY=IX+NNZ ;
      DATA1(IY)=ACOUTSTO(JT) ;
      PUT FILE(PRINT1) EDIT(DATA1(IY)) (F(6,0));
      IF ACOUTSTO(JT)>CMAX THEN CMAX=ACOUTSTO(JT) ;
      DATA3(IY)=AMASSOUT(JT) ;
```

```
      DATA4(IY)=AMASSCUT(JT)/1000.0 ;
      PUT FILE(PRINT2) EDIT(DATA4(IY)) (F(6,0));
      IF AMASSOUT(JT)>CMAX2 THEN CMAX2=AMASSOUT(JT) ;
         YM1=YM1+AMASSOUT(JT) ;
         YM2=YM2+AMASSOUT(JT)*AMASSCUT(JT) ;
         XF1=XF1+AGCCFS(JT) ;
         XF2=XF2+AQOCFS(JT)*ACOCFS(JT) ;
         XC1=XC1+ACCUTSTO(JT) ;
         XC2=XC2+ACCUTSTO(JT)*ACCUTSTO(JT) ;
         XCFW1=XCFW1+AMASSOUT(JT)*4.45 ;
      END ;
      END ;
P1:   IF DIS¬=1.0 THEN DO ;
      NNZ=0 ;
      DO JT=1 TO NW WHILE(AQOCFS(JT)>0.1) ;
      NNZ=NNZ+1 ;
      IY=IX+NNZ ;
      DATA1(IY)=ACOUTSTO(JT) ;
      PUT FILE(PRINT1) EDIT(DATA1(IY)) (F(6,0));
      IF ACOUTSTO(JT)>CMAX THEN CMAX=ACOUTSTO(JT) ;
      DATA3(IY)=AMASSOUT(JT) ;
      DATA4(IY)=AMASSCUT(JT)/1000.0 ;
      PUT FILE(PRINT2) EDIT(DATA4(IY)) (F(6,0));
      IF AMASSOUT(JT)>CMAX2 THEN CMAX2=AMASSCUT(JT) ;
      YM1=YM1+AMASSCUT(JT) ; YM2=YM2+AMASSOUT(JT)*AMASSOUT(JT) ;
      XF1=XF1+AQOCFS(JT) ; XF2=XF2+AQOCFS(JT)*AQOCFS(JT) ;
      XC1=XC1+ACOUTSTO(JT) ; XC2=XC2+ACOUTSTO(JT)*ACOUTSTO(JT) ;
      XCFW1=XCFW1+AMASSOUT(JT)*4.45 ; END ;
      END ;
      DO IR=1 TO NN ;
      YMM1=YMM1+MASSIN(IR) ; YMM2=YMM2+MASSIN(IR)*MASSIN(IR) ;
      XCCFW1=XCCFW1+CINSTO(IR)*FSTOR(IR) ;
      IF NN=1 THEN DO ;
```

```
     XCCFWI=CINSTO(IR) ; END ;
     XCC1=XCC1+CINSTO(IR) ; XCC2=XCC2+CINSTO(IR)*CINSTO(IR) ;
     XFF1=XFF1+FSTCR(IR) ;
     XFF2=XFF2+FSTCR(IR)*FSTOR(IR) ;
     END ;
             /* END OF STORAGE MODEL */
           IF I=52 THEN DO ;
    PUT PAGE ;
    PUT SKIP(2) LIST('*** STORM SEWER & COMB.SEWER OVERFLOW ***');
    PUT SKIP LIST('***         INTO STORAGE , CFS           ***');
    PUT EDIT((FSTOR(IM) DO IM=1 TO NN)) (SKIP,(10)F(9,1));
    PUT SKIP(2) LIST('*** MIXED BOD CONC. INTO STORAGE , MG/L ***');
    PUT EDIT((CINSTO(IM) DO IM=1 TO NN)) (SKIP,(10)F(9,1));
    PUT SKIP(2) LIST('*** BOD MASS INTO STORAGE , LBS/HOUR ***');
    PUT EDIT((MASSIN(IM) DO IM=1 TO NN)) (SKIP,(10)F(9,1));
      IF DIS¬=1.0 THEN DO ;
    PUT SKIP(3) EDIT('*** PREVIOUS MIXED BASIN CONC.=',CONCNN(I),' MG/
L ***') (A,F(7,2),A);   END ;
    IF VV=1.0 THEN DO ;
    PUT SKIP(2) EDIT('*** VARIABLE VOLUME RESERVOIR ***') (COL(25),A);
    PUT SKIP EDIT('*** CUBIC FEET ***') (COL(32),A);
    PUT EDIT((VOL(IM) DO IM=1 TO NN)) (SKIP,(10)F(12,1));
    PUT SKIP EDIT('***    ******    ***') (COL(32),A);
    IF DIS=1.0 THEN PUT SKIP(2) EDIT('*** LONGITUDINAL DISPERSION COEF
FICIENT IN STORAGE =',ES,' SQUARE FEET/HOUR ***') (A,F(12,1),A);
    PUT SKIP(2) EDIT('*** BOD DECAY RATE , <SETTLING> :',KSTO,' ***')
                (A,F(6,2),A);  IF DIS¬=1.0 THEN DO;
    PUT SKIP(2) EDIT('*** BASIN OUTFLOW @ T=0 :',QINIT,' CFS *** DWH @
 T=0 :',DWT(I),' HOURS ** CONC @ T=0 :',CNCTO(I),' MG/L ***')
        (A,F(9,1),A,F(4),A,F(7,2),A);
    PUT SKIP EDIT('*** BASIN OUTFLOW @ T=0 :',QOTO(I),' CFH ***') (A,F
(9,1),A);   END ;
    PUT SKIP(2) LIST('*** FLOW OUTPUT FROM STORAGE , CFS ***');
```

```
PUT EDIT((AQOCFS(IM) DO IM=1 TO NW)) (SKIP,(10)F(9,1));
END ;
PUT SKIP(2) LIST('*** MIXED BOD CONC. OUTPUT FROM STORAGE , MG/L *
**');
PUT EDIT((ACOUTSTO(IM) DO IM=1 TO NW)) (SKIP,(10)F(9,1));
PUT SKIP(2) LIST('*** BOD MASS RATE FROM STORAGE , LBS/HOUR ***');
PUT EDIT((AMASSOUT(IM) DO IM=1 TO NW)) (SKIP,(10)F(9,1));
          END ;
XCM(I)=XC1 ;  YMO(I)=YM1 ; XFM(I)=XF1 ;
XCCFWM(I)=XCCFW1 ;
XCS(I)=0.0 ; YSO(I)=0.0 ; XFS(I)=0.0 ;
XCCS(I)=0.0 ; YSSI(I)=0.0 ; XFFS(I)=0.0 ;
  /* FLOW-WEIGHTED AVERAGE OF MEAN CONC. */
      XCFWM(I)=XCFW1/XF1 ;
 YMO(I)=YM1/NNZ ; YM3=YMO(I)*YMO(I)/(NNZ-1) ;
 YSO(I)=SQRT(YM2/(NNZ-1)-NNZ*YM3) ;
 XCM(I)=XC1/NNZ ; XFM(I)=XF1/NNZ ; XC3=XCM(I)*XCM(I)/(NNZ-1) ;
 XF3=XFM(I)*XFM(I)/(NNZ-1) ; XCS(I)=SQRT(XC2/(NNZ-1)-NNZ*XC3) ;
 XFS(I)=SQRT(XF2/(NNZ-1)-NNZ*XF3) ;
YMMI(I)=YMM1/NN ;
XCCM(I)=XCC1/NN ;  XFFM(I)=XFF1/NN ;
IF NN¬=1 THEN DO ;
XCCFWM(I)=XCCFW1/XFF1 ;
XCC3=XCCM(I)*XCCM(I)/(NN-1) ; XFF3=XFFM(I)*XFFM(I)/(NN-1) ;
XFFS(I)=SQRT(XFF2/(NN-1)-NN*XFF3) ;
XCCS(I)=SQRT(XCC2/(NN-1)-NN*XCC3) ;
YMM3=YMMI(I)*YMMI(I)/(NN-1) ; YSSI(I)=SQRT(YMM2/(NN-1)-NN*YMM3) ;
END ;
CVC(I)=0.0 ; CVM(I)=0.0 ; CVF(I)=0.0 ;
CVCC(I)=0.0 ; CVMI(I)=0.0 ; CVFF(I)=0.0 ;
IF YMO(I)¬=0.0 THEN CVM(I)=YSO(I)/YMO(I) ;
IF YMMI(I)¬=0.0 THEN CVMI(I)=YSSI(I)/YMMI(I) ;
IF XCM(I)¬=0.0 THEN CVC(I)=XCS(I)/XCM(I) ;
```

```
      IF XFM(I)¬=0.0 THEN CVF(I)=XFS(I)/XFM(I) ;
      IF XFFM(I)¬=0.0 THEN CVFF(I)=XFFS(I)/XFFM(I) ;
      IF XCCM(I)¬=0.0 THEN CVCC(I)=XCCS(I)/XCCM(I) ;
      X4=X4+XCM(I) ; X5=X5+XCS(I) ; XI4=XI4+XCCM(I) ; XI5=XI5+XCCS(I) ;
      Y4=Y4+YMO(I) ; Y5=Y5+YSO(I) ; YI4=YI4+YMMI(I) ; YI5=YI5+YSSI(I) ;
      W4=W4+XCFWM(I) ; WI4=WI4+XCCFWM(I) ;
      ZZ4=ZZ4+XFM(I) ; ZZ5=ZZ5+XFS(I) ; ZZI4=ZZI4+XFFM(I) ;
      ZZI5=ZZI5+XFFS(I) ; XCVF=XCVF+CVF(I) ; XCVFI=XCVFI+CVFF(I) ;
      XCVM=XCVM+CVM(I) ; XCVMI=XCVMI+CVMI(I) ;
      XCVC=XCVC+CVC(I) ; XCVCI=XCVCI+CVCC(I) ;
              IF I=52 THEN DO ;
      IF VV=1.0 THEN PUT SKIP(2) EDIT('*** DETENTION TIME =',ADT,' HOURS
 ***') (A,F(3),A);
      PUT SKIP(5) EDIT('*** XCCM =',XCCM(I),' MG/L *** YMMI =',YMMI(I),'
 LBS/HR ***') (A,F(9,1),A,F(9,1),A);
      PUT EDIT('*** FLOW WEIGHTED BODC IN =',XCCFWM(I),' MG/L ***')
              (X(5),A,F(9,1),A);
      PUT SKIP EDIT('*** XCCS =',XCCS(I),' MG/L *** YSSI =',YSSI(I),' LB
S/HR ***') (A,F(9,1),A,F(9,1),A);
      PUT SKIP EDIT('*** CVCC =',CVCC(I),' *** CVMI =',CVMI(I),' ***')
          (A,F(9,2),A,F(9,2),A);
      PUT SKIP EDIT('*** XFFM =',XFFM(I),' CFS ***') (A,F(9,1),A);
      PUT SKIP EDIT('*** XFFS =',XFFS(I),' CFS ***') (A,F(9,1),A);
      PUT SKIP EDIT('*** CVFF =',CVFF(I),' ***') (A,F(9,2),A);
      PUT SKIP(5) EDIT('*** XCM =',XCM(I),' MG/L *** YMO =',YMO(I),' LBS
/HR ***') (A,F(9,1),A,F(9,1),A);
      PUT EDIT('*** FLOW WEIGHTED BODC OUT =',XCFWM(I),' MG/L ***')
              (X(5),A,F(9,1),A);
      PUT SKIP EDIT('*** XCS =',XCS(I),' MG/L *** YSO =',YSO(I),' LBS/HR
***') (A,F(9,1),A,F(9,1),A);
      PUT SKIP EDIT('*** CVC =',CVC(I),' *** CVM =',CVM(I),' ***')
          (A,F(9,2),A,F(9,2),A);
      PUT SKIP EDIT('*** XFM =',XFM(I),' CFS ***') (A,F(9,1),A);
```

```
PUT SKIP EDIT('*** XFS =',XFS(I),' CFS ***') (A,F(9,1),A);
PUT SKIP EDIT('*** CVF =',CVF(I),' ***') (A,F(9,2),A);
    /* PLOT OF FLOW -- INTO AND OUT OF -- STORAGE */
PUT PAGE ;
PUT EDIT('*** FLOW INTO STORAGE , CFS ***') (SKIP,COL(50),A);
DO IP=1 TO NN ;
TTIME(IP)=IP ;
CIN(IP)=FSTOR(IP) ;
END ;
CALL PLTC (TTIME,CIN,NN) ;
PUT PAGE ;
PUT EDIT('*** FLOW FROM STORAGE , CFS ***') (SKIP,COL(50),A);
DO IP=1 TO NW ;
TTIME(IP)=IP ;
CIN(IP)=AQOCFS(IP) ;
END ;
CALL PLTC (TTIME,CIN,NW) ;
                /* END OF PLOT */
    /* PLOT OF EOD CONC. --INTO AND OUT OF -- STORAGE */
PUT PAGE ;
PUT EDIT('*** EOD CONC. INTO STORAGE, MG/L ***') (SKIP,COL(45),A);
DO IP=1 TO NN ;
TTIME(IP)=IP ;
CIN(IP)=CINSTO(IP) ;
END ;
CALL PLTC (TTIME,CIN,NN) ;
PUT PAGE ;
PUT EDIT('*** EOD CONC. FROM STORAGE, MG/L ***') (SKIP,COL(45),A);
DO IP=1 TO NW ;
TTIME(IP)=IP ;
CIN(IP)=ACOUTSTO(IP) ;
END ;
CALL PLTC (TTIME,CIN,NW) ;
```

```
                  /* END OF PLOT */
       /* PLOT OF BOD MASS RATE --INTO AND OUT OF-- STORAGE */
    PUT PAGE ;
    PUT EDIT('*** BOD MASS RATE INTO STORAGE, LBS/HR ***')
            (SKIP,COL(40),A);
    DO IP=1 TO NN ;
    TTIME(IP)=IP ; CIN(IP)=MASSIN(IP) ; END ;
    CALL PLTC (TTIME,CIN,NN) ;
    PUT PAGE ;
    PUT EDIT('*** BOD MASS RATE FROM STORAGE, LBS/HR ***')
            (SKIP,COL(40),A);
    DO IP=1 TO NW ;
    TTIME(IP)=IP ; CIN(IP)=AMASSOUT(IP) ; END ;
    CALL PLTC (TTIME,CIN,NW) ;
             END ;
             /* END OF PLOT */
P2:  IC=1 ; ID=0 ; DATA=DATA2 ;
      IX=IX+NNZ ;
    VOLTO=VOLNN ; CONCTO=CONCNN(I+1) ;
    J=I ;
     Z1,Z2,Z3=0 ;
    X1,Y1,X2,Y2,X3,Y3=0 ;
    XC1,XC2,XC3,XF1,XF2,XF3=0 ;
    XCC1,XCC2,XCC3,XFF1,XFF2,XFF3,XCFW1,XCCFW1=0 ;
    YM1,YM2,YM3,YMM1,YMM2,YMM3=0.0 ;
    FLOWX=RRR1 ; BODMX=RRR2 ; BODCX=RRR3 ;
    DATE=RRR4 ; DWHX=RRR5 ; HOUR=RRR6 ;
    G=RRR7 ; PDWFBOD=PDWFBODX ;
    I=I+1 ;
    IF I<N+1 THEN GO TO L2;
    PUT PAGE ;
    PUT SKIP(5) LIST('*** STATISTICS FOR ENTIRE RECORD SERIES ***') ;
    OMFLOW=ZZ4/N ; IMFLOW=ZZI4/N ;
```

```
CMBODM=Y4/N ;  IMBODM=YI4/N ;
OMBODC=X4/N ;  IMBODC=XI4/N ;
OBODCDEV=X5/N ; IBODCDEV=XI5/N ;
OFLOWDEV=ZZ5/N ; IFLOWDEV=ZZI5/N ;
OBODMDEV=Y5/N ; IBODMDEV=YI5/N ;
OFWBODC=W4/N ; IFWBODC=WI4/N ;
PUT SKIP(5) LIST('*** FROM STORAGE ***');
PUT SKIP(3) EDIT('*** MFLOW =',CMFLOW,' CFS ***') (A,F(10,2),A) ;
PUT SKIP EDIT('*** MBODM =',OMBODM,' LBS/HR ***') (A,F(10,2),A) ;
PUT SKIP EDIT('*** BMDEV =',OBODMDEV,' LBS/HR ***') (A,F(10,2),A);
PUT SKIP EDIT('*** FLDEV =',OFLOWDEV,' CFS ***') (A,F(10,2),A) ;
PUT SKIP EDIT('*** MBODC =',OMBODC,' MG/L ***') (A,F(10,2),A);
PUT EDIT('*** FWBODC =',OFWBODC,' MG/L ***') (X(5),A,F(6,2),A);
PUT SKIP EDIT('*** BCDEV =',OBODCDEV,' MG/L ***') (A,F(10,2),A);
    CVFLOW=OFLOWDEV/OMFLOW ; CVBODM=OBODMDEV/OMBODM ;
    CVBODC=OBODCDEV/OMBODC ;
PUT SKIP(5) EDIT('*** CVFLOW =',CVFLOW,'***') (A,F(5,2),A) ;
PUT SKIP EDIT('*** CVBODM =',CVBODM,'***') (A,F(5,2),A) ;
PUT SKIP EDIT('*** CVBODC =',CVBODC,'***') (A,F(5,2),A);
PUT SKIP(5) LIST('*** INTO STORAGE ***');
PUT SKIP(3) EDIT('*** MFLOW =',IMFLOW,' CFS ***') (A,F(10,2),A);
PUT SKIP EDIT('*** MBODM =',IMBODM,' LBS/HR ***') (A,F(10,2),A);
PUT SKIP EDIT('*** BMDEV =',IBODMDEV,' LBS/HR ***') (A,F(10,2),A);
PUT SKIP EDIT('*** FLDEV =',IFLOWDEV,' CFS ***') (A,F(10,2),A);
PUT SKIP EDIT('*** MBODC =',IMBODC,' MG/L ***') (A,F(10,2),A);
PUT EDIT('*** FWBODC =',IFWBODC,' MG/L ***') (X(5),A,F(6,2),A);
PUT SKIP EDIT('*** BCDEV =',IBODCDEV,' MG/L ***') (A,F(10,2),A);
    CVFLOW=IFLOWDEV/IMFLOW ; CVBODM=IBODMDEV/IMBODM ;
    CVBODC=IBODCDEV/IMBODC ;
PUT SKIP(5) EDIT('*** CVFLOW =',CVFLOW,' ***') (A,F(5,2),A);
PUT SKIP EDIT('*** CVBODM =',CVBODM,' ***') (A,F(5,2),A);
PUT SKIP EDIT('*** CVBODC =',CVBODC,' ***') (A,F(5,2),A);
IF VV¬=1.0 THEN DO ;
```

```
      ASADT=ASADT/N ;
      PUT SKIP(5) EDIT('*** ANNUAL STORM AVERAGE DETENTION TIME =',ASADT
,' HOURS ***') (A,F(4),A) ; END ;
      IF VV=1.0 THEN PUT SKIP(5) EDIT('*** MAXIMUM STORAGE VOLUME USED =
',VMAX,' CUBIC FEET ***') (A,F(13,1),A) ;
      PUT SKIP(3) EDIT('*** MAXIMUM OUTFLOW =',QMAX,' CFS ***')
                  (A,F(9,1),A);
      AVBODSEP=XMSEP/XMSFL ; AVBODCOM=XMCOM/XMCFL ;
      XMCFL=XMCFL*3600.0 ;
      PUT SKIP(5) EDIT('*** AVERAGE BOD CONC. IN SEP. SEWER =',AVECDSEP,
' MG/L ***') (A,F(8,2),A) ;
      PUT SKIP(3) EDIT('*** AVERAGE EOD CONC. IN COMB. SEWER=',AVECDCOM,
' MG/L ***') (A,F(8,2),A) ;
      PUT SKIP(3) EDIT('*** TOTAL CCMB.SEWER FLOW =',XMCFL,' CF/YR ***')
                 (A,F(15,2),A) ;
       PUT PAGE ;
       PUT SKIP(5) EDIT('***** BOD CONC. , MG/L *****') (COL(35),A);
       PUT SKIP(3) EDIT('*** MAXIMUM =',CMAX,' MG/L ***') (COL(35),A,F(7
,2),A) ;
       PUT SKIP(2) LIST('    ');
       PUT EDIT((DATA1(IM) DO IM=1 TO IY)) (SKIP,(15)F(8,2));
       PUT SKIP(5) EDIT('*** TOTAL NO. =',IY,' ***') (COL(30),A,F(5),A);
       PUT PAGE ;
       PUT SKIP(5) EDIT('***** BOD MASS RATE , LBS/HR *****') (COL(30),A
) ;    PUT SKIP(3) EDIT('*** MAXIMUM =',CMAX2,' LBS/HR ***')
                  (COL(30),A,F(10,2),A);
       PUT SKIP(2) LIST('    ');
       PUT EDIT((DATA3(IM) DO IM=1 TO IY)) (SKIP,(10)F(12,2));
          /** PLOTTING SUBROUTINE **/
PLTC: PROCEDURE (O1,O2,NZ) ;
      DCL O(2,80) FLOAT;
      DCL (O1(80),O2(80)) FLOAT;
      DCL NZ FIXED DECIMAL (4);
```

```
DCL LINE (0:100)CHAR(1);
DCL (IBOT2,ITOP2,IBOT1,ITOP1,C1TOP,C2TOP) FLOAT;
DCL (C1BOT,C2BOT,VSCALE,HSCALE) FLOAT;
O=0 ;
DO IK=1 TO NZ ;
O(1,IK)=O1(IK) ; O(2,IK)=O2(IK) ;
END ;
DO II=1 TO NZ-1; DO J=II+1 TO NZ;
IF O(1,II)>O(1,J) THEN DO K=1 TO 2;
Q=O(K,II); O(K,II)=O(K,J); O(K,J)=Q;
END; END; END;
C1TOP,C2TOP=0. ; C1BOT,C2BOT=0.;
DO J=1 TO NZ ;
IF O(1,J)>C1TOP THEN C1TOP=O(1,J);
IF O(2,J)>C2TOP THEN C2TOP=O(2,J);
END;
ITOP2=CEIL(C2TOP/10.)*10. ; IBOT2=FLOOR(C2BOT/10.)*10. ;
IBOT1=FLOOR(C1BOT/10.)*10. ; ITOP1=CEIL(C1TOP/10.)*10. ;
VSCALE=(ITOP2-IBOT2)/100. ;
HSCALE=(ITOP1-IBOT1)/70. ;
L=1 ;
LINE='-' ;
PUT EDIT((X DO X=IBOT2 TO ITOP2 BY VSCALE*10.))
          (SKIP(2),COL(11),(11)F(10,1));
DO J=0 TO 100 BY 10 ; LINE(J)='+' ; END ;
IF FLOOR((O(1,L)-IBOT1)/HSCALE+0.5)=0 THEN DO ;
LINE((O(2,L)-IBOT2)/VSCALE+0.5)='*' ; L=L+1 ;
END;
PUT EDIT(IBOT1,LINE)(COL(10),F(8,2),COL(19),(101)A);
DO K=1 BY 1 TO 70 ;
IF K=32 THEN PUT LIST('      TIME,');
IF K=34 THEN PUT SKIP LIST('          HOURS');
NF=FLOOR(K/7)*7 ;
```

```
TK=K*HSCALE+IBOT1;
LINE=' '; IF NF=K THEN DO ; LINE(0)='+';
PUT EDIT(TK)(COL(10),F(9,1)); END;
          ELSE LINE(0)='I';
NXTPOINT: NLINE=(O(1,L)-IBOT1)/HSCALE+0.5 ;
IF NLINE=K THEN DO ;
NCD=(O(2,L)-IBOT2)/VSCALE+0.5 ;
LINE(NCD)='*';
IF FLOOR(NLINE)=70 THEN LINE(NCD)='*';
L=L+1; GO TO NXTPOINT ; END ;
PUT EDIT(LINE)(COL(19),(101)A) ;
END ;
    /* END OF PLOT */
END PLTC ;
END STOR ;
```

APPENDIX II

WELL-MIXED CONSTANT VOLUME AND RECEIVING WATER DIGITAL COMPUTER MODEL

```
STOR:PROC OPTIONS(MAIN)   ;
      DCL 1 DATA2,
          2 RR1,
            (3 RRR1,3 RRR2,3 RRR3) FLOAT,
          2 RR2,
            (3 RRR4,3 RRR5,3 RRR6,3 RRR7,3 RRR8,3 RRR9,3 RR10) FLOAT;
      DCL 1 DATA CONTROLLED,
          2 READ1,
            (3 FLOWX,3 BODMX,3 BODCX) FLOAT,
          2 READ2,
            (3 DATE,3 DWHX,3 HOUR,3 Q,3 RBOD,3 RT,3 RDO) FLOAT;
      DCL (N,T) FLOAT;
      DCL KK FIXED DECIMAL (6)  ;
      DCL (X1,X2,X3,X4,Y1,Y2,Y3,Y4,X5,Y5) FLOAT;
      DCL (XM(58),YM(58),XS(58),YS(58),CVX(58),CVY(58)) FLOAT;
      DCL (FLOW1(100),BODM1(100),BODC1(100)) FLOAT;
      DCL (RBODM(100),RFLOW(100)) FLOAT ;
      DCL (TDET(58),XCFWM(58),XCCFWM(58),ASACT,VMAX) FLOAT;
      DCL (Z1,Z2,Z3,ZZ4,ZZ5) FLOAT;
      DCL (ZM(58),ZS(58),CVZ(58)) FLOAT;
      DCL (COUT(50),KS,V,K1,ST,RR,RELRATE) FLOAT;
      DCL (XC1,XC2,XC3,XCM(58),XCS(58),CVC(58)) FLOAT;
      DCL (K1T,ATOT,ASEP,ACOM) FLOAT;
      DCL (PCTTRT(3),TD(35),PCT(3),KSTO,KST(35),DWFLO) FLOAT;
      DCL (DWF,X,XC(35,3,3),DO(35),FLO(35),DC(35,3,3)) FLOAT;
      DCL (DOCONC(35,3,3),TE(35),DT(35,3,3),DXCONC(35,3,3)) FLOAT;
      DCL (TC(35,3,3),F(35),LO(35,3,3),K2T(35),U(35),VS(35)) FLOAT;
      DCL (TPCT(3),TRTPCT(3)) FLOAT;
      DCL (QSEP(35),QCOMB(35)) FLOAT;
      DCL (PBODCOMB(35),CONCCOMB(35),COMBLO(35)) FLOAT;
      DCL (LOC(35,3,3),MASSIN(35),CINSTO(35),COUTSTO(35)) FLOAT;
      DCL (SEPLO(35),MASSOUT(35),CSFLO(35),DATA1(449,3,3)) FLOAT;
      DCL (TTIME(35),FSTOR(35),CIN(35)) FLOAT;
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```
      DCL NN FIXED DECIMAL (4);
      DCL (ALPHA,VV,VOL(35),QO(35),QOCFS(35)) FLOAT;
      DCL (XFM(58),KSED(58),XFS(58),CVF(58),ADT,TSET,XF1,XF2,XF3) FLOAT;
      DCL (VOLNN,QINIT,INITVCL,QOTO(58),DWT(58),VOLTO) FLOAT;
      DCL (XFFM(58),XFFS(58),CVFF(58),XFF1,XFF2,XFF3) FLOAT;
      DCL (XCC1,XCC2,XCC3,XCCM(58),XCCS(58),CVCC(58)) FLOAT;
      DCL (INITCONC,CONCTO,CNCTO(59),CONCNN(59)) FLOAT ;
      DCL (YMO(58),YMMI(58),YSO(58),YSSI(58),CVM(58),CVMI(58)) FLOAT;
      DCL (IMFLOW,IMBOCM,IMBODC,IBODCDEV,IFLOWDEV,IBODMDEV) FLOAT;
      DCL (ACOUTSTO(35),AMASSOUT(35),AQOCFS(35)) FLOAT;
      PUT PAGE ; PUT SKIP(5) LIST('    ');
      GET LIST(INITVCL,INITCONC,ALPHA,VV,RR,RELRATE) COPY ;
      GET LIST(N,T,V,K1,KS,ST) COPY ;
      GET LIST(ATOT,ASEP,ACOM,PDWFBODX,CWF) COPY ;
      GET LIST(PCTTRT,DWFLO,X,PCT) COPY ;
      GET LIST(FFLBS) COPY ;
      OPEN FILE(BCD) STREAM INPUT;
      OPEN FILE(PRINT1) STREAM OUTPUT;
      IC,ID,IL,X1,Y1,X2,Y2,X3,Y3,X4,X5,Y4,Y5=0.0 ;
      XI4,XI5,YI4,YI5,ZZI4,ZZI5,W4,WI4=0.0 ;
      XCVF,XCVFI,XCVM,XCVMI,XCVC,XCVCI=0.0 ;
      FLOW1=0 ; BODM1=0 ; RBODM=0 ; RFLOW=0 ;
      VOLTO=INITVOL ; DWT=0.0 ; QOTO=0.0 ;
      CONCTO=INITCONC ; CNCTO=0.0 ;
      BODC1=0 ; Z1,Z2,Z3,ZZ4,ZZ5=0 ;
      PDWFBOD=0.0 ; IDET=0.0 ; QSEP=0.0 ; QCOMB=0.0 ;
      PCTTRT=1.-PCTTRT ; ASADT,VMAX=0.0 ;
      PCT=1.-PCT ; D=X/5280.0 ;
   /* STATISTICS OF INDEPENDENT EVENTS */
      J=1 ; IN=0 ; IX=0 ; IY=0 ;
EVNT: I=1 ;
      IE=1 ;
  RD: ALLOCATE DATA;
```

```
   GET FILE(BOD) EDIT(READ1) (F(9,2),F(9,1),F(9,1));
   GET LIST(READ2);
   IC=IC+1 ; IL=IL+1 ;
   IF IL=1 THEN DWT(IL)=DWHX ;
   IF DWHX>T & IL¬=1 THEN DO ;
         IE=IE+1 ;
         DWT(IE)=DWHX ;
         GO TO CNT ;
            END ;
 /* INSERTING ZERC MASS AND FLOW RATE FOR DW HOURS */
               /* WITHIN RUNOFF EVENTS */
   IF IL=1 THEN GC TO L2 ;
   IF IC=1 THEN GC TO L2;
   IF DWHX=0 THEN GO TO L2 ;
   ICWH=DWHX ;
   DC II=1 TO IDWH ;
   IS=II+J-1 ;
   FLOW1(IS)=0 ; GSEP(IS)=0.0 ; GCOMB(IS)=0.0 ;
   BCDM1(IS)=0 ;
   BCDC1(IS)=0 ;
   END ;
   J=J+IDWH ;
   IN=IN+IDWH ;
L2: FLOW1(J)=FLOWX;
   BCDM1(J)=BCDMX;
   BODC1(J)=EODCX;
   QSEP(J)=FLOW1(J)*ASEP/ATOT ;
   QCOMB(J)=FLOW1(J)*ACOM/ATOT ;
               /* END */
   IF IC¬=I THEN ID=ID+DWHX;
   X1=X1+FLOW1(J) ; Y1=Y1+BODM1(J) ; Z1=Z1+BODM1(J)*4.45 ;
   X2=X2+FLOW1(J)*FLOW1(J) ; Y2=Y2+BODM1(J)*BCDM1(J) ;
         J=J+1 ; IN=IN+I ;
```

```
     IF IL=342 THEN GC TC CNT;
     GC TO RD ;
CNT: DATA2=DATA ;
     DO II=1 TO IC-1 ; FREE DATA ; END ;
     IF IL=342 THEN IC=IC+1 ;
     NN=IC+ID-1 ;
     KK=2*(NN+1)/3 ;
         XM(I)=X1 ; YM(I)=Y1 ;
         IF VV¬=1.0 THEN TDET(I)=V/(X1*3600.0) ;
         XS(I)=0 ; YS(I)=0 ;
         ZM(I)=Z1/X1 ;
     IF NN=1 THEN GC TO S ;
     XM(I)=X1/NN ; YM(I)=Y1/NN ;
     ZM(I)=Z1/X1 ;
     X3=XM(I)*XM(I)/(NN-1) ; Y3=YM(I)*YM(I)/(NN-1) ;
     XS(I)=SQRT(X2/(NN-1)-NN*X3) ; YS(I)=SQRT(Y2/(NN-1)-NN*Y3) ;
     IF VV¬=1.0 THEN DO ; TDET(I)=V/(XM(I)*3600.0) ;
     ASADT=ASADT+TDET(I) ; END ;
  S: CVX(I)=0 ; CVY(I)=0 ;
     IF XM(I)¬=0 THEN CVX(I)=XS(I)/XM(I) ;
     IF YM(I)¬=0 THEN CVY(I)=YS(I)/YM(I) ;
SER: DO K=1 TO KK ;
     S1,S2,S3,S4,S5,Z1,Z2,Z3,Z4,Z5=0 ;
     L=K-1 ;
     DC JJ=L+1 TO NN ;
     S1=S1+BODM1(JJ) ; Z1=Z1+FLOW1(JJ) ;
     S4=S4+BODM1(JJ)*BODM1(JJ) ; Z4=Z4+FLOW1(JJ)*FLOW1(JJ) ;
     END ;
     M=NN-L ;
     DC J=1 TO M ;
     S2=S2+BODM1(J) ; Z2=Z2+FLOW1(J) ;
     S3=S3+BODM1(J)*BCDM1(J+L) ; Z3=Z3+FLOW1(J)*FLOW1(J+L) ;
     S5=S5+BODM1(J)*BODM1(J) ; Z5=Z5+FLOW1(J)*FLOW1(J) ;
```

```
      END ;
      SS1=S5-(S2**2)/M ;  SS2=S4-(S1**2)/M ;
      SS3=Z5-(Z2**2)/M ;  SS4=Z4-(Z1**2)/M ;
      RBODM(K)=0 ; RFLOW(K)=0 ;
      IF SS1¬=0 & SS2¬=0 THEN DO ;
      RBODM(K)=(S3-S2*S1/M)/(SQRT(SS1)*SQRT(SS2)) ;
      END;
      IF SS3¬=0 & SS4¬=0 THEN DO ;
      RFLOW(K)=(Z3-Z2*Z1/M)/(SQRT(SS3)*SQRT(SS4)) ;
      END;
      END SER ;
               IF NN>19 | I=1 THEN DO ;
      PUT PAGE ;
      PUT SKIP(3) EDIT('***** RUNOFF EVENT NO.=',I,' *****') (A,F(5),A);
      PUT SKIP(2) LIST('***** FLOW RATE INPUT , CFS *****');
      PUT EDIT((FLOWI(IJ) DO IJ=1 TO NN)) (SKIP,(10)F(9,1));
      PUT SKIP(2) LIST('***** BOD MASS RATE INPUT , LBS/HOUR *****');
      PUT EDIT((BODMI(IJ) DO IJ=1 TO NN)) (SKIP,(10)F(9,1));
      PUT SKIP(2) LIST('***** BOD CONC. , MG/L *****');
      PUT EDIT((BODCI(IJ) DO IJ=1 TO NN)) (SKIP,(10)F(9,1));
      PUT SKIP(2) EDIT('*** XM =',XM(I),' CFS *** YM =',YM(I),' LBS/HR *
**') (A,F(9,1),A,F(9,1),A);
      PUT SKIP EDIT('*** XS =',XS(I),' CFS *** YS =',YS(I),' LBS/HR ***'
) (A,F(9,1),A,F(9,1),A);
      PUT SKIP EDIT('*** CVX =',CVX(I),' *** CVY =',CVY(I),' ***')
(A,F(5,2),A,F(5,2),A);
      PUT SKIP(2) LIST('*** FLOW WEIGHTED STORMWATER BOD CONC. ***') ;
      PUT SKIP(2) EDIT('*** ZM =',ZM(I),' MG/L ***') (A,F(9,1),A);
      PUT SKIP(5) LIST('***********************************************
*********');
      PUT SKIP LIST('*** CALCULATED LAG-K SERIAL CORRELATION COEFFICIENT
S ***');
      PUT SKIP LIST('************************************************
```

```
*****');
      PUT SKIP(2) LIST('*** RHO(K) FCR FLOW RATE ***');
      PUT SKIP LIST('       ');
      PUT EDIT((RFLOW(L) DC L=1 TO KK)) (SKIP,X(5),(5)F(8,2));
      PUT SKIP(5) LIST('*** RHO(K) FOR BOD MASS FLOW RATE ***');
      PUT SKIP LIST('       ');
      PUT EDIT((RBODM(L) CC L=1 TC KK)) (SKIP,X(5),(5)F(8,2));
      NC=IC-1 ;
      PUT SKIP(3) EDIT('*** IL =',IL,' *** NC =',NC,' *** NN =',NN,' ***
') (A,F(4),A,F(3),A,F(3),A);
      PUT SKIP EDIT('*** IN =',IN,' ***') (A,F(4),A);
   /* LAG-ZERO CROSS-CORRELATION OF INPUTS */
      S1,S2,S3,S4,S5=C ;
      Z1,Z2,Z3=0 ;
      DC II=1 TC NN ;
      S1=S1+FLOW1(II)*BODM1(II);
      Z1=Z1+BODC1(II)*FLOW1(II);
      S2=S2+FLOW1(II);
      S3=S3+BODM1(II);
      Z3=Z3+BODC1(II);
      Z2=Z2+BODC1(II)*BODC1(II);
      S4=S4+FLOW1(II)*FLOW1(II);
      S5=S5+BODM1(II)*BODM1(II);
      END;
      DEN1=S4-(S2**2)/NN;
      DEN2=S5-(S3**2)/NN;
      DEN3=Z2-(Z3**2)/NN;
      RZEROWQ=0 ; RZEROCQ=0 ;
      IF DEN1¬=0 & DEN2¬=0 THEN DO;
      RZEROWQ=(S1-S2*S3/NN)/(SQRT(DEN1)*SQRT(DEN2));
      END ;
      IF DEN1¬=0 & DEN3¬=0 THEN DC ;
      RZEROCQ=(Z1-S2*Z3/NN)/(SQRT(DEN1)*SQRT(DEN3));
```

```
      END ;
      PUT SKIP(5) EDIT('***** RZEROWQ =',RZERCWQ,' *****') (A,F(5.2),A);
      PUT SKIP EDIT('***** RZEROCQ =',RZEROCQ,' *****') (A,F(5.2),A);
              END ;
          /* RECEIVING WATER MODEL */
      XC1,XC2,XC3,XF1,XF2,XF3=0.0 ;
      XCC1,XCC2,XCC3,XFF1,XFF2,XFF3,XCFW1,XCCFW1=0.0 ;
      YM1,YM2,YM3,YMM1,YMM2,YMM3=0.0 ;
 REC: DO IR=1 TC NN ;
      DWFSEP=ASEP*DWF/ATOT ;
      DWFCOMB=ACOM*DWF/ATOT ;
      SEPLO(IR)=BODC1(IR) ;
      IF ASEP=0.0 THEN SEPLO(IR)=0.0 ;
      PBODCOMB(IR)=BCDC1(IR)*QCOMB(IR)*0.2248+ACOM*PDWFBODX/ATOT ;
      CSFLO(IR)=QCOMB(IR)+DWFCOMB ;
      FF=0.0 ; CCNCCCMB(IR)=0.0 ;
      IF CSFLO(IR)¬=0.0 THEN DC ;
      FF=FFLBS*ACOM*DWT(I) ; IF IR¬=1 THEN CONCCOMB(IR)=PBCDCOMB(IR)/(CS
FLO(IR)*3600.0) ; IF IR=1 THEN CCNCCOMB(IR)=(PBODCOMB(IR)+FF)/(CSFLO(IR
)*3600.0) ;
      END ;
      CCMBLO(IR)=CONCCCMB(IR)*16016.60 ;
      K1T=K1*1.047**(RT-20) ;
      CS=14.652-0.41022*RT+0.0079910*(RT**2.)-0.000077774*(RT**3.);
      CC=CS-RDO ; IF CS<RDO THEN CO=0.0 ;
      FLO(IR)=Q+DWF+QSEP(IR)+QCOMB(IR) ;
      CCNST=2.302585*4.1435*(1.024**(RT-20)) ;
      K2T(IR)=CONST/(FLO(IR))**0.57 ;
      DO(IR)=Q*CO/FLC(IR) ;
      Z=5760.0 ;
      U(IR)=Z*(FLO(IR))**0.03 ;
      F(IR)=K2T(IR)/K1T ;
      VS(IR)=U(IR)/3600.0 ;
```

```
TE(IR)=0.0 ; IF U(IR)>0. THEN TE(IR)=X/U(IR) ;
    /* STORAGE OF STORM SEWER FLOWS AND COMBINED */
          /* SEWER OVERFLOWS */
    /* CONSTANT VOLUME -- DISCRETE INPUTS */
S1.S2=0.0 ;
CINSTO(IR)=0.0 ; COUTSTO(IR)=0.0 ;
FSTOR(IR)=0.0 ;
FSTOR(IR)=QSEP(IR)+QCOMB(IR) ;
IF FSTOR(IR)¬=0.0 THEN DO ;
CINSTO(IR)=(QSEP(IR)*BODC1(IR)+QCOMB(IR)*COMBLO(IR))/FSTOR(IR) ;
END ;
   /* MG/L =4.45 LBS/HR/CFS */
MASSIN(IR)=FSTOR(IR)*CINSTO(IR)*0.224719 ;
KSED(I)=0.0 ; IF I¬=1 THEN DO ; TSET=0.0 ;
IF VV¬=1.0 & TDET(I-1)<10.0 THEN TSET=DWT(I)-TDET(I-1) ;
IF TSET<0.0 THEN TSET=0.0 ; IF TSET>20.0 THEN TSET=20.0 ;
KSED(I)=-LOG(0.90+0.10*EXP(-0.7*TSET)) ; END ;
CNCTO(I)=CONCTO*EXP(-KS*DWT(I)-KSED(I)) ;
TD(IR)=0.0 ; KST(IR)=0.0 ;
IF FSTOR(IR)¬=0.0 THEN TD(IR)=V/(FSTOR(IR)*3600.0) ;
KST(IR)=-LOG(0.90+0.10*EXP(-0.7*TD(IR))) ;
S1=FSTOR(IR)*3600.0/V+KS+KST(IR) ;
S2=FSTOR(IR)*3600.0*CINSTO(IR)/(FSTOR(IR)*3600.0+(KS+KST(IR))*V) ;
COUTSTO(IR)=S2*(EXP(S1)-1)*EXP(-S1)+CNCTO(I)*EXP(-S1) ;
IF IR-1 ¬=0 THEN DO ;
COUTSTO(IR)=S2*(EXP(S1)-1)*EXP(-S1)+COUTSTO(IR-1)*EXP(-S1) ;
END ;
 IF COUTSTO(IR)<0.0 THEN COUTSTO(IR)=0.0 ;
 ACOUTSTO(IR)=S2-(S2-CNCTO(I))*(1.-EXP(-S1))/S1 ;
 IF IR¬=1 THEN ACOUTSTO(IR)=S2-(S2-COUTSTO(IR-1))*(1.-EXP(-S1))/S1
 ; IF ACOUTSTO(IR)<0.0 THEN ACOUTSTO(IR)=0.0 ;
IF I=1 THEN CONCNN(I)=INITCONC ;
   CONCNN(I+1)=COUTSTO(IR) ;
```

```
 P1: IF VV¬=1.0 THEN DO ;
     QOCFS(IR)=FSTOR(IR) ; AQOCFS(IR)=QOCFS(IR) ;
     AMASSOUT(IR)=QOCFS(IR)*ACOUTSTO(IR)*0.224719 ;
      END ;
      YM1=YM1+AMASSOUT(IR) ; YM2=YM2+AMASSOUT(IR)*AMASSOUT(IR) ;
      XF1=XF1+AQOCFS(IR) ; XF2=XF2+AQOCFS(IR)*AQOCFS(IR) ;
      YMM1=YMM1+MASSIN(IR) ; YMM2=YMM2+MASSIN(IR)*MASSIN(IR) ;
      XC1=XC1+ACOUTSTO(IR) ; XC2=XC2+ACOUTSTO(IR)*ACOUTSTO(IR) ;
      IF NN¬=1 THEN DO ; XCFW1=XCFW1+AMASSOUT(IR)*4.45 ;
      XCCFW1=XCCFW1+CINSTO(IR)*FSTOR(IR) ; END ;
      IF NN=1 THEN DO ;
      XCFW1=ACOUTSTO(IR) ; XCCFW1=CINSTO(IR) ; END ;
      XCC1=XCC1+CINSTO(IR) ; XCC2=XCC2+CINSTO(IR)*CINSTO(IR) ;
      XFF1=XFF1+FSTOR(IR) ;
      XFF2=XFF2+FSTOR(IR)*FSTOR(IR) ;
               /* END OF STORAGE MODEL */
        DO MJ=1 TO 3 ; DO ML=1 TO 3 ;
     IF VV=1.0 & ST¬=0.0 THEN DO ;
     FLO(IR)=Q+DWF+QOCFS(IR) ; K2T(IR)=CONST/(FLO(IR))**0.57 ;
     DO(IR)=Q*CO/FLO(IR) ; U(IR)=Z*(FLO(IR))**0.03 ; F(IR)=K2T(IR)/K1T;
     VS(IR)=U(IR)/3600.0 ; TE(IR)=0.0 ;
     IF U(IR)>0. THEN TE(IR)=X/U(IR) ;  END ;
     LC(IR,MJ,ML)=(C*RBOD+DWFSEP*PCTTRT(MJ)*DWFLO+DWFCOMB*PCTTRT(MJ)*CO
MBLO(IR)+QOCFS(IR)*ACOUTSTO(IR)*PCT(ML))/FLO(IR) ;
     IF ST=0.0 THEN DO ;
     LC(IR,MJ,ML)=(C*RBOD+DWFSEP*PCTTRT(MJ)*DWFLO+DWFCOMB*PCTTRT(MJ)*CO
MBLO(IR)+(QSEP(IR)*SEPLO(IR)+QCOMB(IR)*COMBLO(IR))*PCT(ML))/FLO(IR) ;
     END ;
     LCC(IR,MJ,ML)=LC(IR,MJ,ML)/(1-EXP(-5*24*K1T)) ;
          END ; END ;
          /* METHOD OF MODIFIED STREETER-PHELPS */
        DO MJ=1 TO 3 ; DO ML=1 TO 3 ;
     F1=F(IR)-1. ; TC(IR,MJ,ML)=0.0 ; XC(IR,MJ,ML)=0.0 ;
```

```
      IF LOC(IR,MJ,ML)¬=0.0 THEN RO=DO(IR)/LOC(IR,MJ,ML) ;
      IF F1¬=0.0 THEN IF LOC(IR,MJ,ML)¬=0.0 THEN IF RO>=0.0 & RO<=(1./F(
IR))  THEN DO ;
      TC(IR,MJ,ML)=(LOG(F(IR)*(1.-F1*DO(IR)/LOC(IR,MJ,ML))))/(K1T*F1) ;
      XC(IR,MJ,ML)=U(IR)*TC(IR,MJ,ML)/5280.0 ;
      DC(IR,MJ,ML)=(K1T*LOC(IR,MJ,ML)/K2T(IR))*EXP(-K1T*TC(IR,MJ,ML)) ;
      END ;
             ELSE DC(IR,MJ,ML)=DO(IR) ;
             ELSE DC(IR,MJ,ML)=DO(IR) ;
             ELSE DC(IR,MJ,ML)=LOC(IR,MJ,ML)*EXP(DO(IR)/LOC(IR,MJ,ML)-1.);
      DOCONC(IR,MJ,ML)=CS-DC(IR,MJ,ML) ;
      IF DC(IR,MJ,ML)>CS THEN DOCONC(IR,MJ,ML)=0.0 ;
      F2=K2T(IR)-K1T ;
      IF F2¬=0. THEN DT(IR,MJ,ML)=(EXP(-K1T*TE(IR))-EXP(-K2T(IR)*TE(IR))
)*K1T*LOC(IR,MJ,ML)/F2+DO(IR)*EXP(-K2T(IR)*TE(IR)) ;
      ELSE DT(IR,MJ,ML)=K1T*LOC(IR,MJ,ML)*TE(IR)*EXP(-K1T*TE(IR))+DO(IR)
*EXP(-K2T(IR)*TE(IR)) ;
      DXCONC(IR,MJ,ML)=CS-DT(IR,MJ,ML) ;
      IF DT(IR,MJ,ML)>CS THEN DXCONC(IR,MJ,ML)=0.0 ;
      IY=IX+IR ;
      DATA1(IY,MJ,ML)=DOCONC(IR,MJ,ML) ;
      PUT FILE(PRINT1) EDIT(DATA1(IY,MJ,ML)) (F(5,2));
           END ; END ;
      END REC ;
                  IF NN>19 | I=1 THEN DO ;
      PUT PAGE ;
      PUT SKIP(2) LIST('*** STORM SEWER & COMB.SEWER OVERFLOW ***');
      PUT SKIP LIST('***           INTO STORAGE , CFS            ***');
      PUT EDIT((FSTOR(IM) DO IM=1 TO NN)) (SKIP,(10)F(9,1));
      PUT SKIP(2) LIST('*** MIXED BOD CONC. INTO STORAGE , MG/L ***');
      PUT EDIT((CINSTO(IM) DO IM=1 TO NN)) (SKIP,(10)F(9,1));
      PUT SKIP(2) LIST('*** BOD MASS INTO STORAGE , LBS/HOUR ***');
      PUT EDIT((MASSIN(IM) DO IM=1 TO NN)) (SKIP,(10)F(9,1));
```

```
      PUT SKIP(3) EDIT('*** PREVIOUS MIXED BASIN CONC.=',CONCNN(I),' MG/
L ***') (A,F(7,2),A);
      IF VV¬=1.0 THEN DO ;
      PUT SKIP(2) EDIT('*** CONSTANT VOLUME RESERVOIR ***') (COL(5),A);
      PUT SKIP EDIT('*** VOLUME =',V,' CUBIC FT. ***') (A,F(12,1),A);
      PUT SKIP(2) EDIT('*** DWH @ T=0 :',DWT(I),' HOURS ** CONC @ T=0 :'
,CNCTO(I),' MG/L ***') (A,F(4),A,F(7,2),A);
      PUT SKIP(2) EDIT('*** VARIABLE DETENTION TIME , HOURS ***') (COL(5
),A);
      PUT EDIT((TD(IM) DO IM=1 TO NN)) (SKIP,(15)F(6));
      PUT SKIP(2) EDIT('*** BOD DECAY RATE DUE TO SEDIMENTATION ***')
                  (COL(5),A);
      PUT EDIT((KST(IM) DO IM=1 TO NN)) (SKIP,(15)F(6,2));
      PUT SKIP(2) EDIT('*** VARIABLE FLOW THROUGH RESERVOIR , CFS ***')
                  (COL(5),A);
      PUT EDIT((FSTOR(IM) DO IM=1 TO NN)) (SKIP,(10)F(9,1));
      END ;
      PUT SKIP(2) LIST('*** MIXED BOD CONC. OUTPUT FROM STORAGE , MG/L *
**');
      PUT EDIT((ACOUTSTO(IM) DO IM=1 TO NN)) (SKIP,(10)F(9,1));
      PUT SKIP(2) LIST('*** BOD MASS RATE FROM STORAGE , LBS/HOUR ***');
      PUT EDIT((AMASSOUT(IM) DO IM=1 TO NN)) (SKIP,(10)F(9,1));
      PUT SKIP(2) EDIT('***** RECEIVING WATER D.O. @ X , MG/L *****')
          (COL(10),A);
      PUT SKIP EDIT('***** X =',D,' MILES DOWNSTREAM *****') (COL(10),A,
F(6,2),A);
      PUT SKIP EDIT('***** MJ = 2 *** ML = 1 *****') (COL(10),A);
      PUT EDIT((DXCONC(IM,2,I) DO IM=1 TO NN)) (SKIP,(15)F(6,2));
              END ;
      XCM(I)=XCI ; XCFWM(I)=XCFW1 ; YMO(I)=YM1 ; XFM(I)=XF1 ;
      XCCM(I)=XCC1 ; XCCFWM(I)=XCCFW1 ; YMMI(I)=YMM1 ; XFFM(I)=XFF1 ;
      XCS(I)=0.0 ; YSC(I)=0.0 ; XFS(I)=0.0 ;
      XCCS(I)=0.0 ; YSSI(I)=0.0 ; XFFS(I)=0.0 ;
```

```
      IF NN¬=1 THEN DO ;
      YMO(I)=YM1/NN ; YMMI(I)=YMM1/NN ;
      YM3=YMO(I)*YMO(I)/(NN-1) ; YMM3=YMMI(I)*YMMI(I)/(NN-1) ;
      YSO(I)=SQRT(YM2/(NN-1)-NN*YM3) ;
      YSSI(I)=SQRT(YMM2/(NN-1)-NN*YMM3) ;
       /* MEAN CONC.-S ARE FLOW-WEIGHTED AVERAGES */
      XCM(I)=XC1/NN ; XCFWM(I)=XCFW1/XF1 ; XFM(I)=XF1/NN ;
      XC3=XCM(I)*XCM(I)/(NN-1) ; XF3=XFM(I)*XFM(I)/(NN-1) ;
      XCS(I)=SQRT(XC2/(NN-1)-NN*XC3) ; XFS(I)=SQRT(XF2/(NN-1)-NN*XF3) ;
      XCCM(I)=XCC1/NN ; XCCFWM(I)=XCCFW1/XFF1 ; XFFM(I)=XFF1/NN ;
      XCC3=XCCM(I)*XCCM(I)/(NN-1) ; XFF3=XFFM(I)*XFFM(I)/(NN-1) ;
      XFFS(I)=SQRT(XFF2/(NN-1)-NN*XFF3) ;
      XCCS(I)=SQRT(XCC2/(NN-1)-NN*XCC3) ;
      END ;
      CVC(I)=0.0 ; CVM(I)=0.0 ; CVF(I)=0.0 ;
      CVCC(I)=0.0 ; CVMI(I)=0.0 ; CVFF(I)=0.0 ;
      IF YMO(I)¬=0.0 THEN CVM(I)=YSO(I)/YMO(I) ;
      IF YMMI(I)¬=0.0 THEN CVMI(I)=YSSI(I)/YMMI(I) ;
      IF XCM(I)¬=0.0 THEN CVC(I)=XCS(I)/XCM(I) ;
      IF XFM(I)¬=0.0 THEN CVF(I)=XFS(I)/XFM(I) ;
      IF XFFM(I)¬=0.0 THEN CVFF(I)=XFFS(I)/XFFM(I) ;
      IF XCCM(I)¬=0.0 THEN CVCC(I)=XCCS(I)/XCCM(I) ;
      X4=X4+XCM(I) ; X5=X5+XCS(I) ; XI4=XI4+XCCM(I) ; XI5=XI5+XCCS(I) ;
      Y4=Y4+YMO(I) ; Y5=Y5+YSO(I) ; YI4=YI4+YMMI(I) ; YI5=YI5+YSSI(I) ;
      W4=W4+XCFWM(I) ; WI4=WI4+XCCFWM(I) ;
      ZZ4=ZZ4+XFM(I) ; ZZ5=ZZ5+XFS(I) ; ZZI4=ZZI4+XFFM(I) ;
      ZZI5=ZZI5+XFFS(I) ; XCVF=XCVF+CVF(I) ; XCVFI=XCVFI+CVFF(I) ;
      XCVM=XCVM+CVM(I) ; XCVMI=XCVMI+CVMI(I) ;
      XCVC=XCVC+CVC(I) ; XCVCI=XCVCI+CVCC(I) ;
              IF NN>19 | I=1 THEN DO ;
      IF VV¬=1.0 THEN PUT SKIP(2) EDIT('*** AVERAGE DETENTION TIME =',TD
ET(I),' HOURS ***') (A,F(7,2),A);
      IF VV=1.0 THEN PUT SKIP(2) EDIT('*** DETENTION TIME =',ADT,' HOURS
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***') (A,F(3),A);
      PUT SKIP(5) EDIT('*** XCCM =',XCCM(I),' MG/L *** YMMI =',YMMI(I),'
 LBS/HR ***') (A,F(9,1),A,F(9,1),A);
      PUT EDIT('*** FLOW WEIGHTED BODC IN =',XCCFWM(I),' MG/L ***')
              (X(5),A,F(9,1),A);
      PUT SKIP EDIT('*** XCCS =',XCCS(I),' MG/L *** YSSI =',YSSI(I),' LB
S/HR ***') (A,F(9,1),A,F(9,1),A);
      PUT SKIP EDIT('*** CVCC =',CVCC(I),' *** CVMI =',CVMI(I),' ***')
          (A,F(9,2),A,F(9,2),A);
      PUT SKIP EDIT('*** XFFM =',XFFM(I),' CFS ***') (A,F(9,1),A);
      PUT SKIP EDIT('*** XFFS =',XFFS(I),' CFS ***') (A,F(9,1),A);
      PUT SKIP EDIT('*** CVFF =',CVFF(I),' ***') (A,F(9,2),A);
      PUT SKIP(5) EDIT('*** XCM =',XCM(I),' MG/L *** YMC =',YMO(I),' LBS
/HR ***') (A,F(9,1),A,F(9,1),A);
      PUT EDIT('*** FLOW WEIGHTED BODC OUT =',XCFWM(I),' MG/L ***')
              (X(5),A,F(9,1),A);
      PUT SKIP EDIT('*** XCS =',XCS(I),' MG/L *** YSO =',YSO(I),' LBS/HR
***') (A,F(9,1),A,F(9,1),A);
      PUT SKIP EDIT('*** CVC =',CVC(I),' *** CVM =',CVM(I),' ***')
          (A,F(9,2),A,F(9,2),A);
      PUT SKIP EDIT('*** XFM =',XFM(I),' CFS ***') (A,F(9,1),A);
      PUT SKIP EDIT('*** XFS =',XFS(I),' CFS ***') (A,F(9,1),A);
      PUT SKIP EDIT('*** CVF =',CVF(I),' ***') (A,F(9,2),A);
          /* PLOT OF FLOW -- INTO AND OUT OF -- STORAGE */
      PUT PAGE ;
      PUT EDIT('*** FLOW INTO STORAGE , CFS ***') (SKIP,COL(50),A);
      DO IP=1 TO NN ;
      TTIME(IP)=IP ;
      CIN(IP)=FSTOR(IP) ;
      END ;
      CALL PLTC (TTIME,CIN) ;
      PUT PAGE ;
      PUT EDIT('*** FLOW FROM STORAGE , CFS ***') (SKIP,COL(50),A);
```

```
DO IP=1 TO NN ;
IF VV¬=1.0 THEN AQOCFS(IP)=QOCFS(IP) ;
CIN(IP)=AQOCFS(IP) ;
END ;
CALL PLTC (TTIME,CIN) ;
                 /* END OF PLOT */
    /* PLOT OF BOD CONC, --INTO AND OUT OF -- STORAGE */
PUT PAGE ;
PUT EDIT('*** BOD CONC. INTO STORAGE, MG/L ***') (SKIP,COL(45),A);
DO IP=1 TO NN ;
CIN(IP)=CINSTO(IP) ;
END ;
CALL PLTC (TTIME,CIN) ;
PUT PAGE ;
PUT EDIT('*** BOD CONC. FROM STORAGE, MG/L ***') (SKIP,COL(45),A);
DO IP=1 TC NN ;
CIN(IP)=ACOUTSTO(IP) ;
END ;
CALL PLTC (TTIME,CIN) ;
                 /* END OF PLOT */
 PUT PAGE ;
 PUT SKIP(5) EDIT('***** RECEIVING WATER CRITICAL VALUES *****')
               (COL(10),A) ;
 PUT SKIP(3) EDIT('*** CRITICAL DO CONC. , MG/L ***') (COL(5),A);
 PUT EDIT((DOCONC(IM,2,1) DO IM=1 TO NN)) (SKIP,(15)F(7,2)) ;
 PUT SKIP(2) EDIT('*** CRITICAL DISTANCES , MILES ***') (COL(5),A)
 ; PUT EDIT((XC(IM,2,1) DO IM=1 TO NN)) (SKIP,(10)F(8,2)) ;
 PUT SKIP(2) EDIT('*** CRITICAL TIMES , HOURS ***') (COL(5),A);
 PUT EDIT((TC(IM,2,1) DO IM=1 TC NN)) (SKIP,(15)F(6)) ;
 PUT SKIP(2) EDIT('*** STREAM VELOCITY , FT/SEC ***') (COL(5),A);
 PUT EDIT((VS(IM) DO IM=1 TO NN)) (SKIP,(15)F(5,2)) ;
 PUT SKIP(2) EDIT('*** MIXED BOD CONC. IN STREAM , MG/L ***') (COL
(5),A) ; PUT EDIT((LOC(IM,2,1) DO IM=1 TO NN)) (SKIP,(15)F(7,2)) ;
```

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     PUT SKIP(2) EDIT('*** INITIAL DEFICITS , MG/L ***') (COL(5),A);
     PUT EDIT((DO(IM) DO IM=1 TO NN)) (SKIP,(15)F(7,2));
    PUT SKIP(2) EDIT('*** CS =',CS,' MG/L *** K1T =',K1T,' 1/HOUR ***'
) (COL(5),A,F(6,2),A,F(7,2),A);
    PUT SKIP(2) EDIT('*** REAERATION RATES (K2T) , 1/HOUR ***')
                (COL(5),A) ;
    PUT EDIT((K2T(IM) DO IM=1 TO NN)) (SKIP,(10)F(8,2)) ;
           END ;
 P2: IC=1 ; IO=0 ; DATA=DATA2 ;
    VOLTO=VOLNN ; CONCTO=CONCNN(I+1) ;
    IX=IX+NN ;
    RBODM,RFLOW,BODM1,FLOW1=0 ; J=1 ;
    BODC1=0 ; Z1,Z2,Z3=0 ;
    X1,Y1,X2,Y2,X3,Y3=0 ;
    XC1,XC2,XC3,XF1,XF2,XF3=0 ;
    XCC1,XCC2,XCC3,XFF1,XFF2,XFF3,XCFW1,XCCFW1=0 ;
    YM1,YM2,YM3,YMM1,YMM2,YMM3=0.0 ;
    FLOWX=RRR1 ; BODMX=RRR2 ; BODCX=RRR3 ;
    DATE=RRR4 ; DWHX=RRR5 ; HOUR=RRR6 ;
    G=RRR7 ; PDWFBOD=PDWFBODX ;
    I=I+1 ;
    IF I<N+1 THEN GO TO L2;
    PUT PAGE ;
    PUT SKIP(5) LIST('*** STATISTICS FOR ENTIRE RECORD SERIES ***') ;
    OMFLOW=ZZ4/N ; IMFLOW=ZZI4/N ;
    OMBODM=Y4/N ; IMBODM=YI4/N ;
    OMBODC=X4/N ; IMBODC=XI4/N ;
    OBODCDEV=X5/N ; IBODCDEV=XI5/N ;
    OFLOWDEV=ZZ5/N ; IFLOWDEV=ZZI5/N ;
    OBODMDEV=Y5/N ; IBODMDEV=YI5/N ;
    OFWBODC=W4/N ; IFWBODC=WI4/N ;
    PUT SKIP(5) LIST('*** FROM STORAGE ***');
    PUT SKIP(3) EDIT('*** MFLOW =',OMFLOW,' CFS ***') (A,F(10,2),A) ;
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      PUT SKIP EDIT('*** MBODM =',OMBCDM,' LBS/HR ***') (A,F(10,2),A) ;
      PUT SKIP EDIT('*** BMDEV =',OBODMDEV,' LBS/HR ***') (A,F(10,2),A);
      PUT SKIP EDIT('*** FLDEV =',CFLCWDEV,' CFS ***') (A,F(10,2),A) ;
      PUT SKIP EDIT('*** MBODC =',OMBODC,' MG/L ***') (A,F(10,2),A);
      PUT EDIT('*** FWBODC =',CFWBODC,' MG/L ***') (X(5),A,F(6,2),A);
      PUT SKIP EDIT('*** BCDEV =',OBODCDEV,' MG/L ***') (A,F(10,2),A);
          CVFLOW=XCVF/N ; CVBODM=XCVM/N ; CVBODC=XCVC/N ;
      PUT SKIP(5) EDIT('*** CVFLOW =',CVFLOW,'***') (A,F(5,2),A) ;
      PUT SKIP EDIT('*** CVBODM =',CVBODM,'***') (A,F(5,2),A) ;
      PUT SKIP EDIT('*** CVBODC =',CVBODC,'***') (A,F(5,2),A);
      PUT SKIP(5) LIST('*** INTO STORAGE ***');
      PUT SKIP(3) EDIT('*** MFLOW =',IMFLOW,' CFS ***') (A,F(10,2),A);
      PUT SKIP EDIT('*** MBODM =',IMBODM,' LBS/HR ***') (A,F(10,2),A);
      PUT SKIP EDIT('*** BMDEV =',IBODMDEV,' LBS/HR ***') (A,F(10,2),A);
      PUT SKIP EDIT('*** FLDEV =',IFLCWDEV,' CFS ***') (A,F(10,2),A);
      PUT SKIP EDIT('*** MBODC =',IMBODC,' MG/L ***') (A,F(10,2),A);
      PUT EDIT('*** FWBODC =',IFWBODC,' MG/L ***') (X(5),A,F(6,2),A);
      PUT SKIP EDIT('*** BCDEV =',IBODCDEV,' MG/L ***') (A,F(10,2),A);
          CVFLOW=XCVFI/N ; CVBODM=XCVMI/N ; CVBODC=XCVCI/N ;
      PUT SKIP(5) EDIT('*** CVFLOW =',CVFLOW,' ***') (A,F(5,2),A);
      PUT SKIP EDIT('*** CVBODM =',CVBODM,' ***') (A,F(5,2),A);
      PUT SKIP EDIT('*** CVBODC =',CVBODC,' ***') (A,F(5,2),A);
      IF VV¬=1.0 THEN DO ;
      ASADT=ASADT/N ;
      PUT SKIP(5) EDIT('*** ANNUAL STORM AVERAGE DETENTION TIME =',ASADT
,' HOURS ***') (A,F(4),A) ; END ;
      IF VV=1.0 THEN PUT SKIP(5) EDIT('*** MAXIMUM STORAGE VOLUME USED =
',VMAX,' CUBIC FEET ***') (A,F(13,1),A) ;
  P3: PUT PAGE ;
      PUT SKIP(2) LIST('          ***** D.O. CONCENTRATION , MG/L *****');
      PUT SKIP LIST('          ');
      PUT EDIT((((DATA1(I,J,L) DO I=1 TO 449) DO J=1 TO 3) DO L=1 TO 3))
 (SKIP,(15)F(8,2));
```

```
PLTC: FROCEDURE (01,02) ;
   /* PLOTTING SUBROLTINE */
      DCL O(2,35) FLCAT;
      DCL (O1(35),O2(35)) FLOAT;
      DCL LINE (0:100)CHAR(1);
      CCL (IBOT2,ITOF2,IBOT1,ITOP1,C1TOP,C2TOP) FLOAT;
      DCL (C1BOT,C2BCT,VSCALE,HSCALE) FLOAT;
      O=0 ;
      DC IK=1 TC NN ;
      C(1,IK)=O1(IK) ; O(2,IK)=O2(IK) ;
      END ;
      DC II=1 TC NN-1; DO J=II+1 TC NN;
      IF O(1,II)>O(1,J) THEN DO K=1 TO 2;
      G=C(K,II); O(K,II)=O(K,J); O(K,J)=Q;
      END; END; END;
      C1TOP,C2TOP=0. ; C1BOT,C2BOT=0.;
      DC J=1 TC NN ;
      IF O(1,J)>C1TOP THEN C1TOP=O(1,J);
      IF O(2,J)>C2TOP THEN C2TOP=O(2,J);
      END;
      ITOP2=CEIL(C2TOP/10.)*10. ; IBOT2=FLOOR(C2BOT/10.)*10. ;
      IBOT1=FLOOR(C1BOT/10.)*10. ; ITOP1=CEIL(C1TOP/10.)*10. ;
      VSCALE=(ITOP2-IBCT2)/100. ;
      HSCALE=(ITOP1-IBOT1)/60. ;
      L=1 ;
      LINE='-' ;
      PUT EDIT((X DO X=IBCT2 TO ITOP2 BY VSCALE*10.))
                (SKIP(2),COL(11),(11)F(10,1));
      DO J=0 TO 10C BY 10 ; LINE(J)='+' ; END ;
      IF FLOOR((O(1,L)-IBOT1)/HSCALE+0.5)=0 THEN DO ;
      LINE((O(2,L)-IBCT2)/VSCALE+0.5)='*' ; L=L+1 ;
      END;
      PUT EDIT(IBOT1,LINE)(COL(10),F(8,2),COL(19),(101)A);
```

```
DO K=1 BY 1 TO 60 ;
IF K=32 THEN PUT LIST('        TIME,');
IF K=34 THEN PUT SKIP LIST('        HOURS');
NF=FLOOR(K/6)*6 ;
TK=K*HSCALE+IBCT1;
LINE=' '; IF NF=K THEN DO ; LINE(0)='+';
PUT EDIT(TK)(COL(10),F(9,1)); END;
          ELSE LINE(0)='I';
NXTPOINT: NLINE=(O(1,L)-IBOT1)/HSCALE+0.5 ;
IF NLINE=K THEN DO ;
NCD=(O(2,L)-IBCT2)/VSCALE+0.5 ;
LINE(NCD)='*';
IF FLOOR(NLINE)=60 THEN LINE(NCD)='*';
L=L+1; GO TO NXTPOINT ; END ;
PUT EDIT(LINE)(COL(19),(101)A) ;
END ;
     /* END OF PLOT */
END PLTC ;
END STOR ;
```

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BIOGRAPHICAL SKETCH

Miguel Angel Medina, Jr. was born on December 9, 1946 in Havana, Cuba. He departed for Spain in 1960, where he stayed briefly before moving to the United States. He received a Bachelor of Civil Engineering degree from the University of Alabama in 1968. In 1969, he was commissioned a second lieutenant in the United States Army Artillery. He served as Intelligence Officer of the 528th Artillery Group in Turkey, and as Assistant Post Engineer at Fort McPherson, Georgia. In 1972, he was awarded a Master of Science in Civil Engineering degree from the University of Alabama. Since that time, Mr. Medina has been pursuing the degree of Doctor of Philosophy in Environmental Engineering Sciences at the University of Florida.

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I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

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